Stonehaven Channel Capacity Study

Draft Report

July 2010

Aberdeenshire Council
Carlton House
Arduthie Road
Stonehaven
AB39 2DP

JBA Office

JBA Consulting
Port Neuk
1 Longcraig Road
South Queensferry
Edinburgh
EH30 9TD

JBA Project Manager

Caroline Anderton BSc MSc CEnv CSci MCIWEM C.WEM

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Contract

This report describes work commissioned by Steve McFarland, on behalf of Aberdeenshire Council, through purchase order NS1128250. Aberdeenshire Council's representative for the contract was Steve McFarland. Caroline Anderton and Marion McMillan of JBA Consulting carried out this work.

Prepared by .... Marion McMillan MA

Analyst

.... Caroline Anderton BSc MSc CEnv CSci

MCIWEM C.WEM

Senior Analyst

Reviewed by .... David Bassett BSc MSc CEnv MCIWEM C.WEM

Divisional Manager

Purpose

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JBA Consulting has no liability regarding the use of this report except to Aberdeenshire Council.

Acknowledgments

Thank you to Steve McFarland and Alison Struthers for providing data and photographs for the November 2009 event.

Thank you to Derek Fraser of SEPA for providing hydrometric data for the Carron gauge.

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Abbreviations

1-D	One Dimensional (modelling)
AMAX	Annual Maximum
DDF	Depth Duration Frequency
FEH	Flood Estimation Handbook
GEV	General Extreme Value Distribution
GL	General Logistic Distribution
JBA	JBA Consulting – Engineers & Scientists
mAOD	metres Above Ordnance Datum
NGR	National Grid Reference
OS	Ordnance Survey
OS NGR	Ordnance Survey National Grid Reference
POT	Peaks Over a Threshold
QMED	Median Annual Flood (with return period 2 years)
SEPA	Scottish Environment Protection Agency

1. Introduction

1.1 Purpose of this study

Heavy rainfall over the period leading up to the 1st November 2009 resulted in extensive flooding to the centre of Stonehaven. During this event the River Carron overtopped its banks upstream of the 'Green Bridge' along Low Wood Road and Carron Terrace. Once out of bank, flooding from overland flow was extensive, causing ponding of water along a number of roads in the town centre.

Figure 1-1: Stonehaven November 2009 - Photos sourced from the BBC News Website

1.2 Study scope

JBA Consulting were asked to provide an assessment of the capacity of the River Carron with the aim of feeding into Aberdeenshire Council's own reports on the November 2009 event. As per our quotation letter of 8 March 2010, the scope of this report is as follows:

- Carry out an initial assessment of capacity from Walker Bridge to the Sea
    - Construct a hydraulic model in InfoWorks RS of the reach of the River Carron from Walker Bridge to the Sea; including the Walker Bridge, Red Bridge, Green Bridge, Fish Passage Weir, White Bridge, Bridgefield Road Bridge and the Footbridge at the beach.
    - Run the hydraulic model for a number of incremental peak flows (including the November 2009 event) to report on the existing capacity at each individual cross section.
    - Discussion of data collected by Aberdeenshire Council during and following the November 2009 event with the aim of using this data to calibrate (or at least verify) the hydraulic model.
    - Initial assessments of the impact of specific channel capacity queries:
        - The impact of the narrowing of the channel at Green Bridge during the 1970s (by increasing the channel width to represent the historical photograph).
        - The capacity restrictions posed by the underside of the Green Bridge (by raising the bridge soffit by 600 mm).
        - The capacity of the Green Bridge prior to the recent removal of sediment (by modelling the data measured by the Council).
        - The effect of the island structure and rock armour on the left bank adjacent to the sewer (assess channel capacity through this reach, u/s and d/s of the island and then remove rock armour from left bank and replace with vertical structure). The effect of removal of the log weir will also be examined.

        - The outlet channel and rock armour at the beach (test the removal of the extension made on the right hand side) - initially modelling indicated that adjustments to this area of the model would not impact on capacity further upstream and therefore this option has been replaced with a combination of raising the trellis framework underneath the Green Bridge and removal of the log weir.
        - Assess the garden section from the White Bridge Footbridge to the Sea.
    - The model downstream boundary, and hence capacity of the lower reach, will be tested using a Low Tide and the 200 year extreme level (previously calculated by JBA for the Council) – this is not a full joint probability analysis.
    - A short report will be produced detailing the methodology used and the results. Recommendations will also be made in terms of future work that the Council may wish to explore.
- Estimate design peak flows
    - Carry out a rating review and incorporate the existing data collected by SEPA on the River Carron into the Flood Estimation Handbook (FEH) analysis.
    - Estimate design peak flows using the most up-to-date FEH methodologies for a range of return period (QMED, 5 year, 10 year, 25 year, 50 year, 75 year, 100 year, 200 year and 1000 year).
    - Provide comment on the channel capacity in terms of return period and the November 2009 event.
    - Flood mapping for the 200 year event along the model reach will be possible once additional details of floodplain levels and modelling of out of bank areas is undertaken.
- Further Hydrometric Data Analysis – Assess hydrological response of the catchment
    - Carry out analysis of the instantaneous data collected by SEPA on the River Carron. Assess the time to peak and volumes over the record length. Discuss how the November 2009 event compares to the general record.
    - Consideration of rainfall data collected by SEPA at Cheyne and Mongour – discussion of these against the River Carron record.

1.3 Study Area - The Carron Catchment

This study concentrates on the reach of the Carron between Walker Bridge (at OS NGR 386690 785489) and the outlet into the North Sea (at OS NGR 387606 785655). The reach includes the Walker Bridge, Red Bridge, Green Bridge, Fish Passage Weir, White Bridge, Bridgefield Road Bridge and the Footbridge at the beach.

The Carron Catchment is approximately 43 km^2 in size and flows from its source in the Brae of Glenbervie (to the south of Fetteresso Forest) in a south easterly direction before passing under the A90 which marks the western boundary of Stonehaven.

The River Carron is gauged by SEPA just below the Red Bridge at OS NGR 386946 785653 (adjacent to model cross section 734). This particular gauge has been in place since 2003. This gauge is not telemetered and is gauged by wading only; therefore this gauge is suitable for reliably assessing low flows only.

2. Hydraulic Modelling Methodology

2.1 Introduction

This section of the report details the modelling methodology adopted and assumptions made. This model has been built to assess the individual capacity of each cross section.

2.2 Survey Data

Topographical survey was carried out by JBA Consulting during March 2010, to Ordnance Survey Datum. This includes 41 cross sections (location of which are shown in Figure 2-1, Figure 2-2 and Figure 2-3) and elevations of all bridges and weirs along the reach.

Figure 2-1: River Carron Cross Sections 1142 to 671

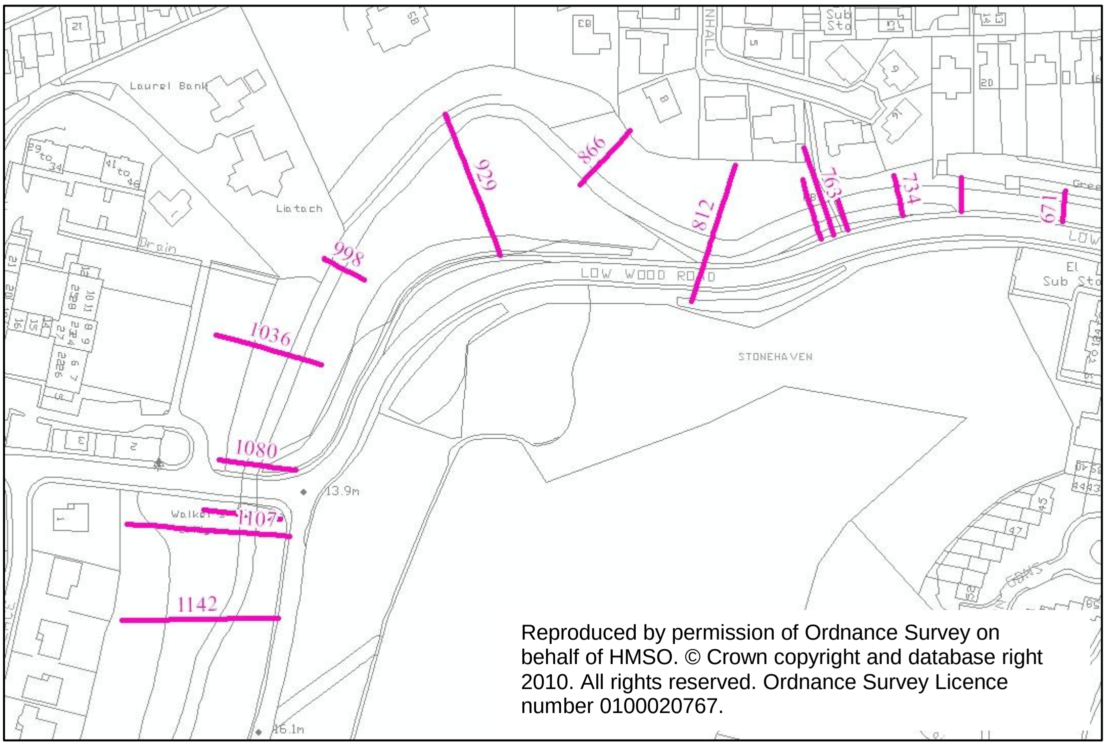


Figure 2-2: River Carron Cross Sections 671 to 295

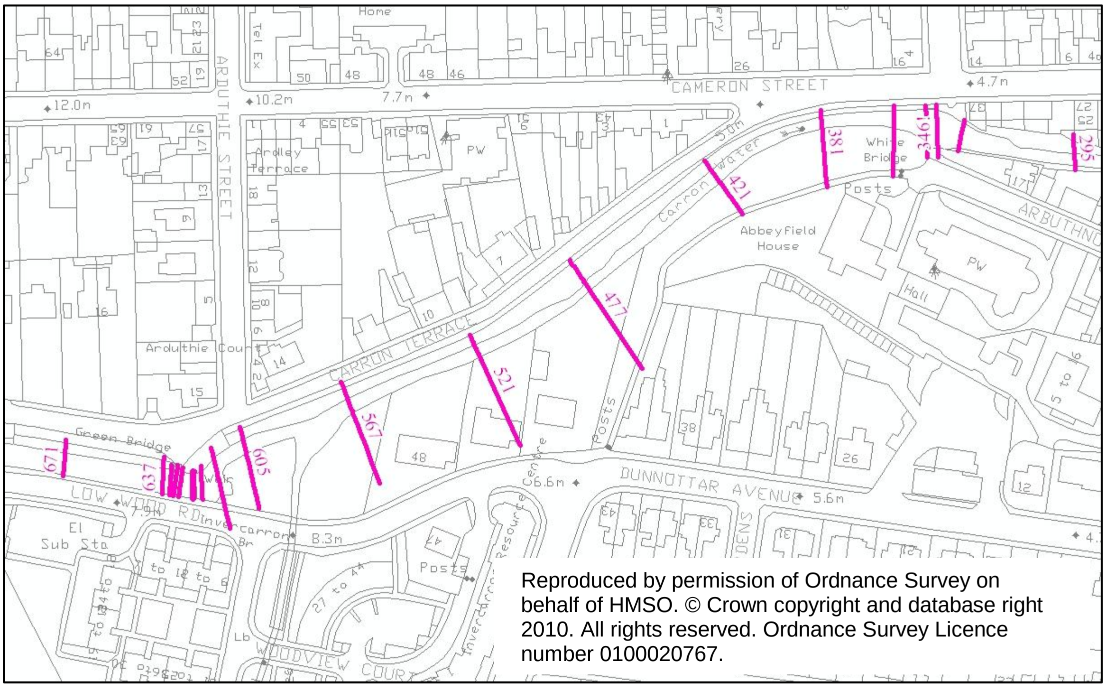
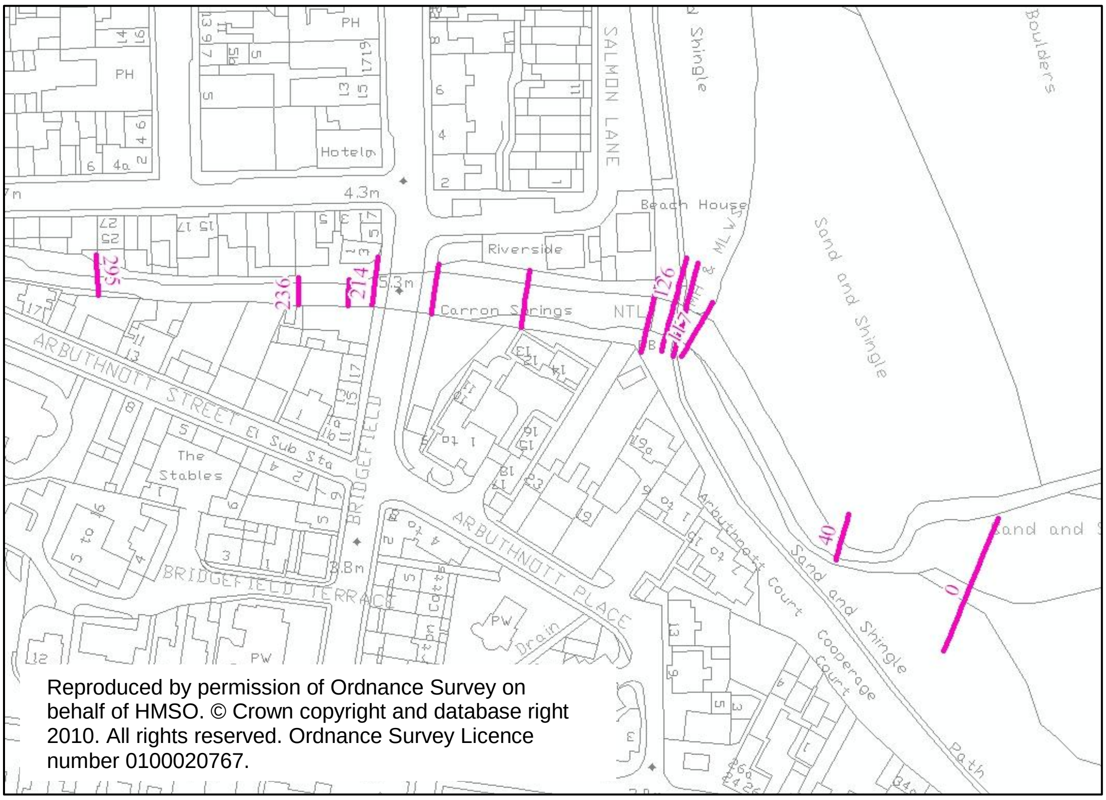


Figure 2-3: River Carron Cross Sections 295 to 0



2.3 Hydraulic Model

A hydraulic model has been constructed in InfoWorks RS version 10.5.6. This model can be used to evaluate both steady and unsteady (hydrodynamic) flows. For the purpose of this study, hydrodynamic modelling was undertaken so that the channel capacity at varying flows could be assessed more rapidly. Hydrodynamic modelling simulates a flood 'event' rather than just a peak flow, and uses inflow hydrographs where flow is variable over time.

The model extends along the reach of the Carron between Walker Bridge (at OS NGR 386690 785489) to the outlet into the North Sea (at OS NGR 387606 785655), an approximate length of 1,142 m. The reach includes the Walker Bridge, Red Bridge, Green Bridge, Fish Passage Weir, White Bridge, Bridgefield Road Bridge and the Footbridge at the beach.

To assess the channel capacity the model has been run for an arbitrary peak flow of 50 m^3/s using a standardised hydrograph shape derived from the gauged data. The attenuation within the model is small as the floodplain is not being modelled and therefore using the unsteady state model is appropriate for assessing channel capacity.

2.3.1 Model parameters

Manning’s 'n' was set as 0.04 for the channel in order to represent the gravel nature of the river bed, and between 0.04 and 0.06 for out of bank areas to represent grassland and rough grass/scattered trees. These values are consistent with standard guidance.

Bridge parapets have been represented by spill units within InfoWorks RS; where trellis work/wire netting is present then this has been included as being 100% blocked and is consistent with floods when mesh structures blind with trash entrained in the flood water. The weir coefficients for each spill unit were set at 1.00.

The log weir at the Green Bridge is represented by a spill unit - therefore allowing the variable levels across the crest to be represented in one unit. A weir coefficient of 1.70 was used, representing a weir that is efficient at allowing water to discharge.

The upstream model boundary was based on a single inflow hydrograph input at the upstream limit of the model. A normal depth, based on the channel gradient in the lower part of the modelled reach, was specified for the downstream boundary condition.

2.4 Model Calibration and Validation

Calibration is necessary to develop confidence in the hydraulic model’s predictions of flood depths and extents, and to test levels of uncertainty and confidence in the parameters used. Calibration is achieved through the use of historic flood data. Ideally, this information is primarily in the form of peak water levels at specific locations, which correspond to peak recorded river flows.

Aberdeenshire Council collected vital post flood survey data during the November 2009 event. This data has been used as reality checking on this modelling and will prove useful if detailed modelling of the floodplain is carried out in the future.

In this case the model is only appropriate for assessing in-channel water levels throughout the model reach. During the topographical survey the stage marker of 1 m at the SEPA staff gauge was levelled in as representing a level of 7.85 mAOD. The right bank (which is slightly lower than constraining ground levels on the left bank) is at a level of 8.92 mAOD and therefore represents an approximate gauged stage of 2.07 m. During the November 2009 event a stage of 2.06 m was recorded and this highlights that the November 2009 event was just within bank at the gauging station.

It is also known that flooding occurred along Low Wood Road (upstream of the Green Bridge) in 2002. At this time the Council report that while water flowed down Low Wood Road it flowed back into the River Carron immediately downstream of the Green Bridge.

2.5 Assumptions

The use of this hydraulic model involves a number of assumptions.

Table 2-1: Model assumptions

Assumption	Comment
Model type	The model is a 1-D hydrodynamic InfoWorks RS model.
Model geometry	This model has been constructed to assess channel capacity at this stage. Overland flow paths or attenuation on the floodplain have not been incorporated. It is normal to incorporated this by using reservoir units connected to the channel via spill units or by carrying out 2D modelling. The model cross sections have been cut to remove low ground outwith the cross sections. Therefore at higher flows glass-walling occurs, where this occurs this can increase predicted water levels. The framework under the Green Bridge has been modelled as being 100% blocked, i.e. the base of the framework represents the underside of the bridge.
Boundary conditions	The model has one inflow point only and this is located at the upstream limits. The downstream boundary is normal depth.
Range of model application	This model has been constructed to assess channel capacity only at this stage and due to glass-walling is not appropriate for floodplain mapping. Additional 2 D modelling can be used to confirm the extent of the floodplain.

3. Study of Channel Capacity

3.1 Existing Channel Capacity

The model was run for a high flow scenario using a normal depth for the downstream boundary condition, and results extracted to show the minimum flow at which the river levels exceed thresholds of flooding at each cross section extracted from the full topographic survey data. These are shown in Table 3-1 below. See Figure 1 for a plan of the approximately locations of assumed thresholds of flooding at each section.

It can be seen that water first starts to exceed threshold levels at cross section 637 immediately upstream of the Green Bridge. This equates to an approximate maximum channel capacity at this cross section of 20 m^3/s. With a slight increase in flow, out of channel flow will commence at cross sections 635 and 671. It is understood that the these locations match those known to have the lowest capacity and where flood water left the channel during the 2009 flood event (estimated to be c. 30 m^3/s). Capacity was also exceeded upstream of the Red Bridge and therefore may have been out of bank at the SEPA gauge (cross section 734). If it can be confirmed that the gauge was being bypassed this may affect the gauged record for this event.

Table 3-1: Base Scenario (existing conditions, normal depth downstream boundary) - Channel capacity at individual cross sections

Node	Left Threshold Level (m AD)	Right Threshold Level (m AD)	Min flow at which threshold level reached (m^3/s)	Rank (based on min flow)
1142	**11.29**	**11.76**	**26.4**	**6**
1107	**11.15**	**11.56**	**29.7**	**9**
1100	13.21	13.18	> 50	
1080	13.24	13.20	> 50	
1036	13.35	13.94	> 50	
998	11.25	13.99	48.6	24
929	10.91	13.73	43.9	22
866	12.40	13.26	> 50	
812	11.97	11.87	> 50	
768	9.34	10.09	34.0	12
763	9.34	9.92	34.0	13
Red Bridge				
757	9.49	9.51	37.1	17
734	9.36	8.92	30.5	10
710	**8.66**	**8.66**	**27.5**	**7**
671	**8.40**	**8.17**	**21.9**	**3**
637	**8.49**	**8.01**	**20.0**	**1**
635	**8.54**	**8.00**	**20.4**	**2**
Green Bridge				
631	8.49	8.09	24.6	4
627	8.23	8.10	25.7	5
Log weir				
624	7.86	8.13	> 50	
605!	7.80	7.25	> 50	
605	6.98	7.25	> 50	
567	6.30	6.37	> 50	
521	5.43	5.93	34.4	14
477	5.23	6.20	35.2	15
421	**5.00**	**4.66**	**29.4**	**8**

Node	Left Threshold Level (m AD)	Right Threshold Level (m AD)	Min flow at which threshold level reached (m^3/s)	Rank (based on min flow)
381	5.31	4.66	39.2	18
357	5.15	4.66	45.1	23
346	5.25	4.60	40.3	20
White Bridge				
334	5.02	5.07	> 50	
295	5.77	5.66	> 50	
236	3.44	5.22	32.4	11
221	3.48	6.82	43.8	21
214	6.23	6.24	> 50	
Bridgefield Road Bridge				
196	3.84	5.86	> 50	
169	3.26	3.53	35.5	16
132	3.29	5.75	39.9	19
126	3.72	3.72	> 50	
117	4.22	4.26	> 50	
40	3.40	4.21	> 50	
0	1.14	2.33	-	
Note: cross sections highlighted in bold are those sites which have a capacity less than that of the November 2009 event.				

3.2 Individual capacity checks

In order to assess the potential for short term remedial works, a number of capacity check scenarios were undertaken as part of this study:

- Scenario 1 - Assessment of the effect of the island structure & rock armour on the left bank next to sewer and removal of log weir
- Scenario 2 - Assessment of the capacity restriction posed by the underside of the Green Bridge
- Scenario 3 - Assessment of the capacity of Green Bridge prior to sediment removal
- Scenario 4 - Assessment of narrowing of the channel at Green Bridge
- Scenario 5 - Test the capacity of the lower reach using low tide level & 200 year extreme sea level
- Scenario 6 - Assessment of removing the log weir and raising trellis framework underneath the Green bridge combined
- Scenario 7 - Assessment of the garden section from the White Bridge to the Sea

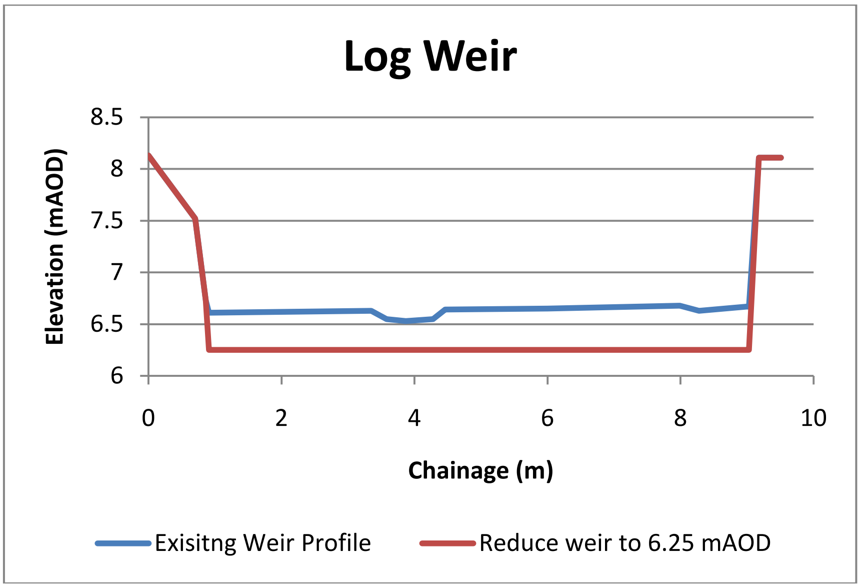
3.2.1 Scenario 1 - Assessment of the effect of the island structure & rock armour on the left bank next to sewer and removal of log weir

Scenario 1a - Reduce weir level to constant level of 6.25 mAOD

The crest elevations of the existing log weir were measured during the JBA topographic survey.

Aberdeenshire Council's design drawing no 4/M/62 Rev 3 'Cross Sections' suggests that the log weir is comprised of a nominal 450 mm diameter timber log. During the JBA topographic survey the log was measured as being c. 430 mm in diameter.

Figure 3-1: Cross Section of Log Weir



The removal of the 'log weir' immediately downstream of the Green Bridge has been represented by reducing the elevation of the current weir (by c. 400 mm) to the lowest surveyed bed level immediately upstream (6.25 mAOD) - see Figure 3-1. This elevation is still some 860 mm above the immediate downstream surveyed bed level of 5.39 mAOD.

The model results suggest that removal of the weir increases the effective flow capacity of the channel for a length of approximately 140 m upstream of the weir. The greatest increase in capacity is found at the weir itself; an additional 10.3 m^3/s can be conveyed immediately upstream of the weir prior to water levels exceeding the threshold level here. The log weir has a considerable effect on capacity because it is the most significant influence on water levels between the Red Bridge and fish passage weir. Removal of the weir results in a significant drop in head upstream of the weir and therefore benefit to upstream capacity.

It would be beneficial to model this scenario with inclusion of overbank areas in order to confirm these results.

Table 3-2: Scenario 1a - Channel Capacities following the removal of the log weir (reduce levels to 6.25mAOD)

Node	Left Threshold Level (m AD)	Right Threshold Level (m AD)	Min flow at which threshold level reached (m^3/s)	Difference in Capacity - Scenario 1 minus Base (m^3/s)
768	9.34	10.09	38.4	4.4
763	9.34	9.92	38.5	4.4
Red Bridge				
757	9.49	9.51	42.0	4.9
734	9.36	8.92	35.4	4.9
710	8.66	8.66	32.6	5.1
671	8.40	8.17	27.8	5.9
637	8.49	8.01	27.2	7.2
635	8.54	8.00	27.5	7.1
Green Bridge				
631	8.49	8.09	33.1	8.5
627	8.23	8.10	36.0	10.3
Log weir				

Removal of the log weir (assuming a low flow notch is retained) will increase fish passage by reducing the overall head difference over the weir, improving the ability of fish to migrate upstream. A low flow notch, the invert of which as far as possible should remain drowned, and a deep pool downstream, would need to be maintained. The velocities through the sections should also be checked in the hydraulic model to ensure they are suitable for fish passage. However, this should represent a positive design consideration and would be likely to require a simple CAR license.

Scenario 1b - Reduce weir level by removing a constant 430 mm from the model weir crest

If on the other hand one assumed that the log has been laid at a slight angle and remove 430 mm from the entire length (with the exception of the low flow notches), as shown in Figure 3-2 below, the effective capacity of the channel immediately upstream of the weir is increases further to 11.2 m^3/s.

Figure 3-2: Cross Section of Log Weir

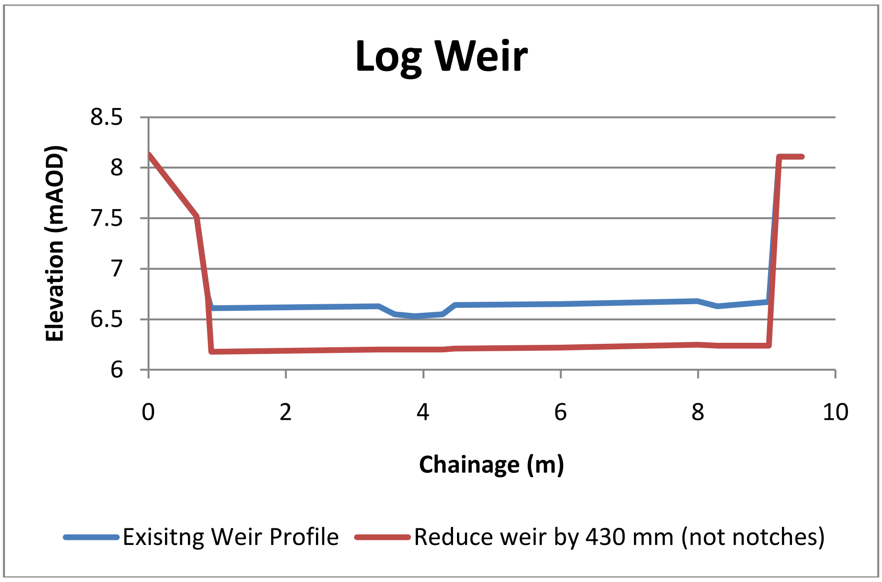


Table 3-3: Scenario 1b - Channel Capacities following the removal of the log weir (reduce weir by 430 mm (excluding notches)

Node	Left Threshold Level (m AD)	Right Threshold Level (m AD)	Min flow at which threshold level reached (m^3/s)	Difference in Capacity - Scenario 1 minus Base (m^3/s)
768	9.34	10.09	38.4	4.4
763	9.34	9.92	39.0	5.0
Red Bridge				
757	9.49	9.51	42.5	5.4
734	9.36	8.92	35.9	5.4
710	8.66	8.66	33.0	5.5
671	8.40	8.17	28.3	6.5
637	8.49	8.01	27.5	7.5
635	8.54	8.00	28.3	7.8
Green Bridge				
631	8.49	8.09	34.0	9.4
627	8.23	8.10	36.9	11.2
Log weir				

Scenario 1c - Testing removal of rock armour on the left bank of the weir

Table 3-4: Scenario 1a plus Rock Armour - Channel Capacities following the removal of the log weir (reduce levels to 6.25mAOD) and channel on left bank increased by c. 0.90m to represent removal of rock armour and replacement with wall

Node	Left Threshold Level (m AD)	Right Threshold Level (m AD)	Min flow at which threshold level reached (m^3/s)	Difference in Capacity - Scenario 1a plus Rock Armour minus Scenario 1a (m^3/s)
768	9.34	10.09	41.0	2.5
763	9.34	9.92	41.6	3.1
Red Bridge				
757	9.49	9.51	45.0	3.0
734	9.36	8.92	38.4	3.0
710	8.66	8.66	35.8	3.2
671	8.40	8.17	30.8	3.0
637	8.49	8.01	30.7	3.5
635	8.54	8.00	31.7	4.2
Green Bridge				
631	8.49	8.09	38.7	5.6
627	8.23	8.10	39.3	3.3
Log weir				

Taking the Scenario 1a model run (where the weir crest has been lowered to a constant level of 6.25 mAOD) and by increasing the lateral width of the channel and log weir by c. 0.9 m and replacing the left bank with a vertical wall, the effective capacity of the channel upstream of the log weir is increased by a further 3 - 5 m^3/s over and above Scenario 1a as shown in Table 3-4).

Comment on the island downstream of the log weir

The base model indicates that flow passes down the main channel up to a flow of c. 5 m^3/s, at which point flow over the high level overflow channel commences. During the November 2009 event water levels were observed to be hitting the nose of the island and causing a standing wave upstream. The island may have been overtopped during the November 2009 event. The base model suggests that the island will not be overtopped until flows exceed 50 m^3/s.

There is a reasonable difference in level between the island and the weir suggesting that the weir and channel bed control water levels upstream in the region of the Green Bridge more than the island downstream. Considering the photographs taken prior to the construction of the new weir this would suggest a large increase in downstream capacity flowing the construction of the new fish passage weir. If the log weir is removed or lowered the control of water levels in this region may change.

3.2.2 Scenario 2 - Assessment of the capacity restriction posed by the underside of the Green Bridge

The framework on the underside of the Green Bridge has a minimum elevation of 7.72 to 7.76 mAOD; this is 240 mm below the right bank elevation of 8.00 mAOD and 540 mm below the underneath of the deck walkway level of 8.30 mAOD. For the purposes of the base model run, the underside of the framework was treated as being the soffit level of the bridge, representing 100% blinding of the framework. During the base model scenario the soffit (underside of the framework) is reached at a flow of 15.48 m^3/s and the right bank threshold is exceeded at a flow of 20.40 m^3/s.

A second model scenario was run with the soffit level raised from 7.76 mAOD to 8.30 mAOD. The soffit in this case is above the threshold of out-of-channel flooding at this location (8.00 mAOD, the level of the right bank). In this scenario the threshold is exceeded at a flow of 21.3 m^3/s and (with glass walling on the right bank) the soffit of the bridge is reached at a flow of 29.05 m^3/s. The limited impact of removing the trellis is as a result of the relatively shallow gradient of the channel immediately upstream of the bridge and the fact that the log weir immediately downstream of the bridge has a greater impact on water levels within the reach than the bridge itself.

Table 3-5: Scenario 2 - Channel Capacities following the modelling of the Green Bridge without the trellis framework

Node	Left Threshold Level (m AD)	Right Threshold Level (m AD)	Min flow at which threshold level reached (m^3/s)	Difference in Capacity - Scenario 2 minus Base (m^3/s)
768	9.34	10.09	40.7	6.8
763	9.34	9.92	41.2	7.2
Red Bridge				
757	9.49	9.51	45.6	8.5
734	9.36	8.92	36.8	6.3
710	8.66	8.66	32.5	5.0
671	8.40	8.17	23.3	1.4
637	8.49	8.01	21.3	1.3
635	8.54	8.00	21.3	0.9
Green Bridge				
631	8.49	8.09	24.1	-0.5
627	8.23	8.10	25.2	-0.6
Log weir				
Note: the channel reach between Section 757 and 627 are subject to out of bank flows on the left and right of the channel. These flows paths are not represented within this scenario.				

The bed profile (and hence water profile) along the model reach between cross section 757 and 635 is reasonably shallow in gradient. The log weir immediately downstream of the Green Bridge is controlling water levels within this whole reach and in addition cross sections 635 and 637 have the lowest thresholds. Therefore the model results for this scenario indicate that the capacity of the cross sections further upstream experience a greater increase in capacity than those immediately upstream of the bridge.

3.2.3 Scenario 3 - Assessment of the capacity of Green Bridge prior to sediment removal

Sediment within the River Carron is highly mobile and significant deposition is known to occur immediately upstream of the Green Bridge. In January 2010 (following the November 2009 flood event) the Council removed the gravel bar from within the channel for a distance of approximately 25m upstream from the Green Bridge. Up to 100 m^3 of sediment is reported to have been removed. The bed levels were not surveyed pre and post gravel removal but the Council did take dip levels from the Green Bridge in relation to the underside of the Green Bridge before the gravel was removed.

Figure 3-3: Cross Section 635 immediately upstream of the Green Bridge

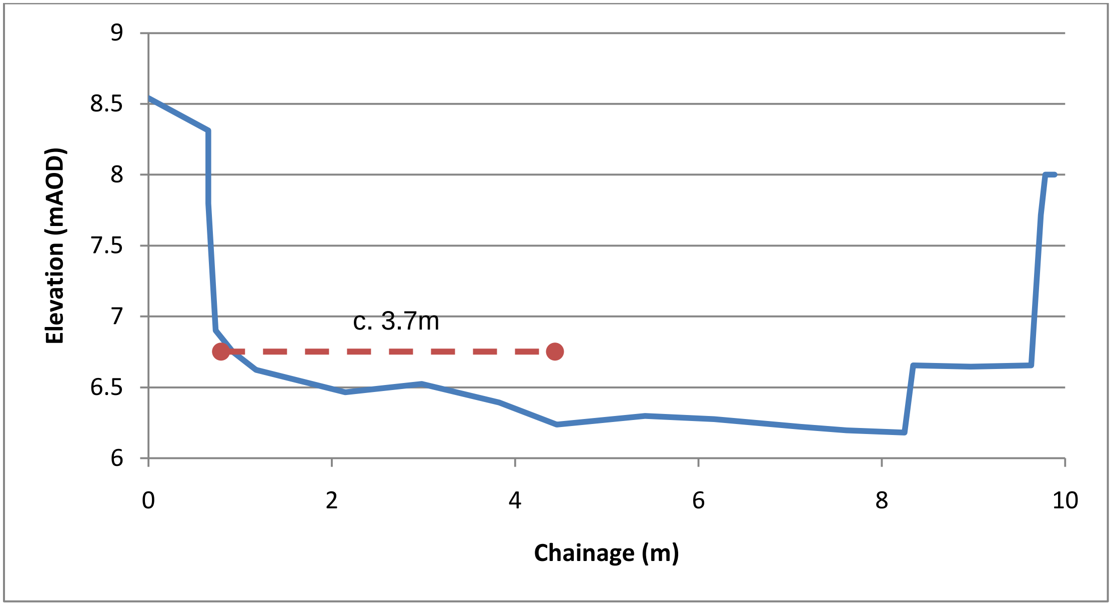


Figure 3-3 shows the cross section immediately upstream of the bridge (cross section 635) surveyed in March 2010. This suggests that the gravel bar would appear to be reforming over a width of approximately 3.7 m in the channel. Therefore if we consider a channel width of 3.7 m and a length of 25 m then this would suggest that to enable the Council to remove 100 m^3 of sediment a depth of around 1.1 m may have been removed. Table 3-6 below would suggest that the bed levels immediately underneath the Green Bridge were measured as being slightly higher in March 2010 than measured prior to the removal of the gradient bar.

Table 3-6: Bed Levels Immediately underneath the Green Bridge

	Left Channel	Mid Channel	Right Channel
(1) Level of the underside of the bridge deck surveyed in March 2010 (mAOD)	8.31	8.35	8.33
(2) Height difference (from underside of deck to bed) measured by the Council (m)	1.75	2.10	2.35
Estimated bed level (1 minus 2) (mAOD)	6.56	6.25	5.98
Current bed level upstream of bridge	6.90	6.24	6.18
Current bed level downstream of bridge	6.74	6.32	6.12
Difference between survey and pre removal level upstream	-0.34	0.01	-0.20
Difference between survey and pre removal level downstream	-0.18	-0.07	-0.14

Figure 3-4: Upstream face of Green Bridge taken while sediment bar submerged (Source: Aberdeenshire Council)

Considering the analysis above, Figure 3-4 and the surveyed sections shown in Figure 3-5 to Figure 3-7, the model geometry was altered (by raising the bed level along the left side of the channel by 630 mm for a distance of 25 m upstream of the Green Bridge) to represent the historic sediment bar upstream of the Green Bridge and to determine its impact on the channel capacity.

Figure 3-5: Cross Section 671! (additional cross section located 10m downstream of 671!)

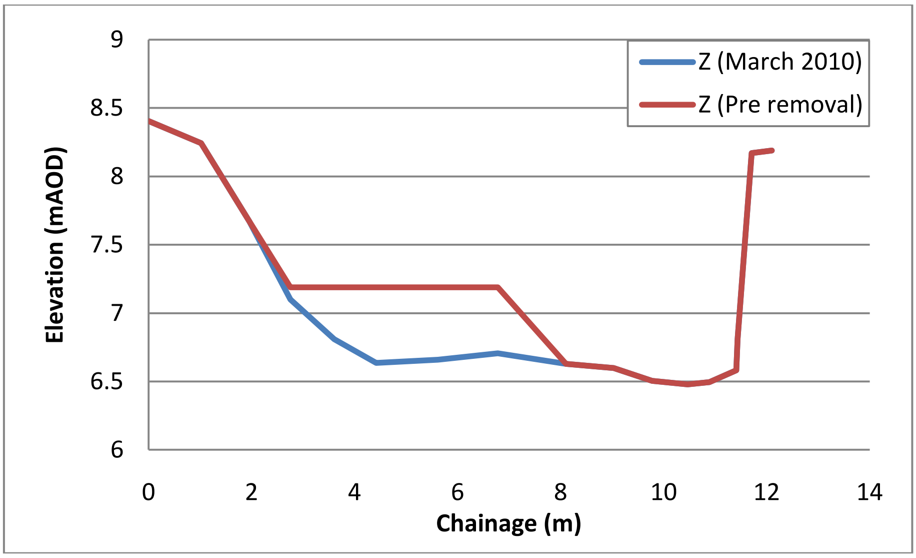


Figure 3-6: Cross Section 637

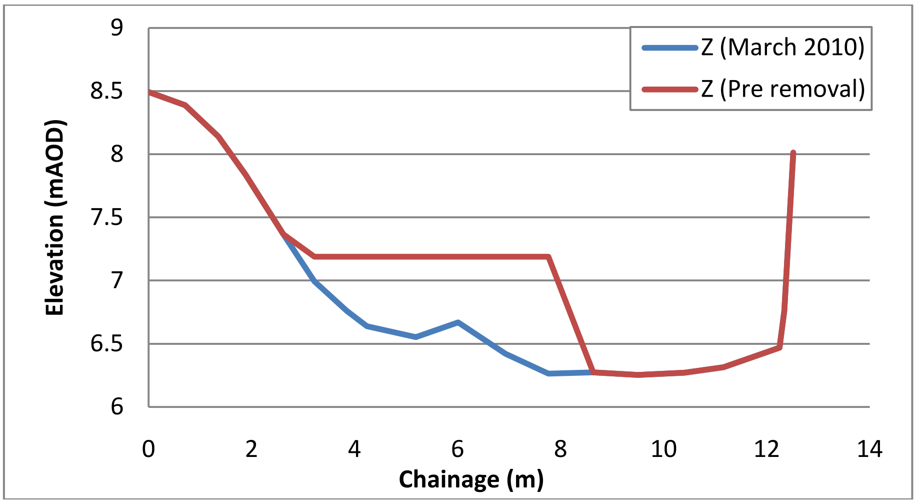


Figure 3-7: Cross Section 635 immediately upstream of the Green Bridge

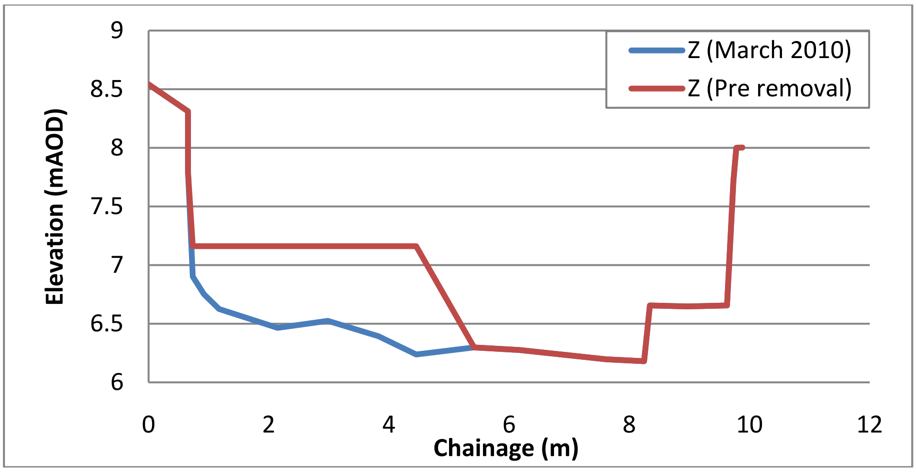


Table 3-7: Scenario 3 - Channel Capacities following the modelling of the addition of the sediment bar upstream of the Green Bridge

Node	Left Threshold Level (m AD)	Right Threshold Level (m AD)	Min flow at which threshold level reached (m^3/s)	Difference in Capacity - Scenario 3 minus Base (m^3/s)
768	9.34	10.09	34.0	0.0
763	9.34	9.92	34.0	0.0
Red Bridge				
757	9.49	9.51	37.1	0.0
734	9.36	8.92	30.1	-0.4
710	8.66	8.66	26.6	-0.9
671	8.40	8.17	19.7	-2.2
637	**8.49**	**8.01**	20.8	0.8
635	**8.54**	**8.00**	21.3	0.9
Green Bridge				
631	8.49	8.09	24.6	0.0
627	8.23	8.10	25.8	0.0
Log weir				

The results from this model scenario are displayed within Table 3-7 and suggest that removal of the sediment bar has resulted in an increased capacity of the channel immediately upstream of the sediment bar (cross sections 734 to 671) at this location by approximately 1 to 2 m^3/s but with an insignificant change at the bar itself (cross sections 637 and 635).

On further investigation of the modelled velocities (Table 3-8) it can be seen that the average channel velocities are higher within the channel with the sediment bar represented, this can be attributed to the sediment bar causing a narrowing of the predominant flow path and hence increased velocities.

Table 3-8: Scenario 3 - Average Channel Velocities following the modelling of the addition of the sediment bar upstream of the Green Bridge

Node	Base Model Channel Velocity (m/s)	Scenario 3 Model Channel Velocity (m/s)	Difference in velocity - Scenario 3 minus Base (m/s)
CS 757	1.83	1.77	-0.06
CS 734	1.62	1.54	-0.09
CS 710	1.57	1.47	-0.10
CS 671	1.55	1.42	-0.12
CS 637	**1.34**	**1.79**	**0.45**
CS 635	**1.45**	**1.92**	**0.47**

3.2.4 Scenario 4 - Assessment of narrowing of the channel at Green Bridge

Aberdeenshire Council supplied a photograph of the Green Bridge which was taken prior to the widening of Low Wood Road from a single track to double width road. A comparison of the photographs below shows that the Green Bridge now has 4.5 railing sections compared to 5 complete railing sections prior to the road widening. Assuming these are all of equal width, the bridge railing was measured as being 9.04 m in width in March 2010 therefore each full section is 2.00 m in width. This would suggest that the river at the Green Bridge and for a reach of c.75 m upstream was narrowed by up to 1.00 m.

To simulate this historical reduction in channel width, cross sections 627 (the log weir) to 710 have been widened by 1 m along the right bank. The results from this model scenario are shown in Table 3-9 below.

Figure 3-8: The Green Bridge Looking Upstream Prior to Widening of the Low Wood Road and existing elevation (Source: Aberdeenshire Council)

Table 3-9: Scenario 4 - Channel Capacities following the modelling of the wider historic channel at the Green Bridge

Node	Left Threshold Level (m AD)	Right Threshold Level (m AD)	Min flow at which threshold level reached (m^3/s)	Difference in Capacity - Scenario 4 minus Base (m^3/s)
929	10.91	13.73	47.5	3.6
866	12.40	13.26	>50.0	-
812	11.97	11.87	>50.0	-
768	9.34	10.09	37.6	3.6
763	9.34	9.92	37.6	3.6
Red Bridge				
757	9.49	9.51	41.2	4.1
734	9.36	8.92	33.9	3.4
710	8.66	8.66	30.4	3.0
671	8.40	8.17	24.1	2.3
637	8.49	8.01	21.9	1.9
635	8.54	8.00	22.5	2.1
Green Bridge				
631	8.49	8.09	26.7	2.1
627	8.23	8.10	28.1	2.4
Log weir				

The modelling of this scenario suggests that the narrowing of the channel at the Green Bridge by 1 m reduced the effective channel capacity by up to 4.1 m^3/s; but by only 1.9 m^3/s at the key cross section 637.

3.2.5 Scenario 5 - Test the capacity of the lower reach using low tide level & 200 year extreme sea level

The base model was run using a normal depth boundary (and therefore independent of any high tide levels). A further model was run using a 200 year extreme sea level of 3.17 mAOD (as provided to the Council by JBA during the Newburgh Flood Study). The model results (shown in Table 3-10 below) indicate that the extreme sea level influences the reach of the river upstream as far as cross section 295, which is located 50 m downstream of the White Bridge. Directly downstream of the Bridgefield Road bridge, the capacity is reduced by up to 15.1 m^3/s with high flow and high sea level conditions.

Table 3-10: Scenario 5 - Channel Capacities following the modelling of the 200 year tide level

Node	Left Threshold Level (m AD)	Right Threshold Level (m AD)	Min flow at which threshold level reached (m^3/s)	Difference in Capacity - Scenario 4 minus Base
295	5.77	5.66	>50	-
236	3.44	5.22	27.0	-5.4
221	3.48	6.82	42.3	-1.5
214	6.23	6.24	>50	-
Bridgefield Road Bridge				
196	3.84	5.86	>50	-
169	3.26	3.53	20.4	-15.1
132	3.29	5.75	26.9	-12.9
126	3.72	3.72	>50	-
117	4.22	4.26	>50	-
40	3.40	4.21	>50	-

3.2.6 Scenario 6 - Removal of log weir and raise framework underneath the Greenbridge

By combining scenarios 1a (removal of log weir to constant 6.25 mAOD and 2 (raising framework underneath the Green Bridge), the model results (Table 3-11 below) suggest that this combination increases the effective flow capacity by 11.2 m^3/s at Cross Section 635.

Table 3-11: Scenario 6 - Channel Capacities following the modelling of the Removal of the Log weir to constant 6.25 mAOD level and Green Bridge without the trellis framework

Node	Left Threshold Level (m AD)	Right Threshold Level (m AD)	Min flow at which threshold level reached (m^3/s)	Difference in Capacity - Scenario 6 minus Base (m^3/s)
768	9.34	10.09	47.0	13.0
763	9.34	9.92	47.3	13.3
Red Bridge				
757	9.49	9.51	41.7	11.2
734	9.36	8.92	37.2	9.7
710	8.66	8.66	29.2	7.3
671	8.40	8.17	29.1	9.1
637	8.49	8.01	29.9	9.5
635	8.54	8.00	41.7	11.2
Green Bridge				

Node	Left Threshold Level (m AD)	Right Threshold Level (m AD)	Min flow at which threshold level reached (m^3/s)	Difference in Capacity - Scenario 6 minus Base (m^3/s)
631	8.49	8.09	32.7	8.2
627	8.23	8.10	36.3	10.6
Log weir				
Note: the channel reach between Section 757 and 627 are subject to out of bank flows on the left and right of the channel. These flows paths are not represented within this scenario.				

3.2.7 Scenario 7 - Assessment of the garden section from the White Bridge to the Sea

The capacity of the channel along the reach from the White Bridge to the outlet to the sea under low tide conditions is reasonably high compared to the reach upstream of the Green Bridge. It should be noted that these capacities are based on threshold levels which are estimated to match the threshold of flooding and do not represent the garden edge - i.e. under high flow conditions the gardens which open out onto the watercourse's left bank will be flooded. In addition this modelling exercise does not account for any future encroachment into the watercourse by the riparian owners.

Flooding to the gardens along the section between the White Bridge and the Bridgefield Bridge commences first at cross section 295 at an approximate flow of 9 m^3/s.

Extended cross sections would be required to test the effect of garden realignment. Removal of garden walls may help reduce flood levels in this reach but are unlikely to increase channel or floodplain capacity unless ground levels are altered and gardens re-profiled.

3.3 Bridge Capacities

The capacities of each of the main bridges have been extracted from the baseline model and are shown in Table 3-12 below. It can be seen from these results that the Walker Bridge, White Bridge and Bridgefield Road bridge have capacities of 50 m^3/s or greater and that the Green Bridge has the lowest capacity at 15.8 m^3/s. However, anecdotal evidence provided to the Council suggests that the Bridgefield Road bridge was running full or even backing up during the November 2009 event[1]. Therefore further modelling of the Bridgefield Bridge may be required to aim to calibrate water levels at the bridge.

Table 3-12: Bridge Capacities extracted from the baseline model - based on water level reaching the soffit

Bridge	Min flow at which threshold level reached (m^3/s)
Walker Bridge	> 50.0
Red Bridge	29.9
Green Bridge	15.8
White Bridge	> 50.0
Bridgefield Bridge	50.0

3.4 November 2009 model run

The model has also been run using the November 2009 event hydrograph (peak flow if 30 m^3/s) to simulate the dynamics of this event within the channel. The flows for the November 2009 event have been estimated by converting the stage measured at the gauge into flow using the model rating. This rating is considered more reliable than that provided by SEPA, which gives an estimate of the November 2009 peak flow at 80 m^3/s, for which it has been extrapolated far beyond its reasonable bounds (See Section 4.3). However, while the actual rating for the November 2009 event may differ from the model rating since the model is based on the current channel geometry and a large quantity of sediment transport is believed

[1] Discussions with Steve McFarland, April 2010

to have taken place during the November 2009 event, it is considered the current best estimate.

The model has been run with a normal depth boundary and therefore is believed to be more conservative at the modelled peak flow, as the tide peaked several hours after the river peaked during the November 2009 event (Table 3-13).

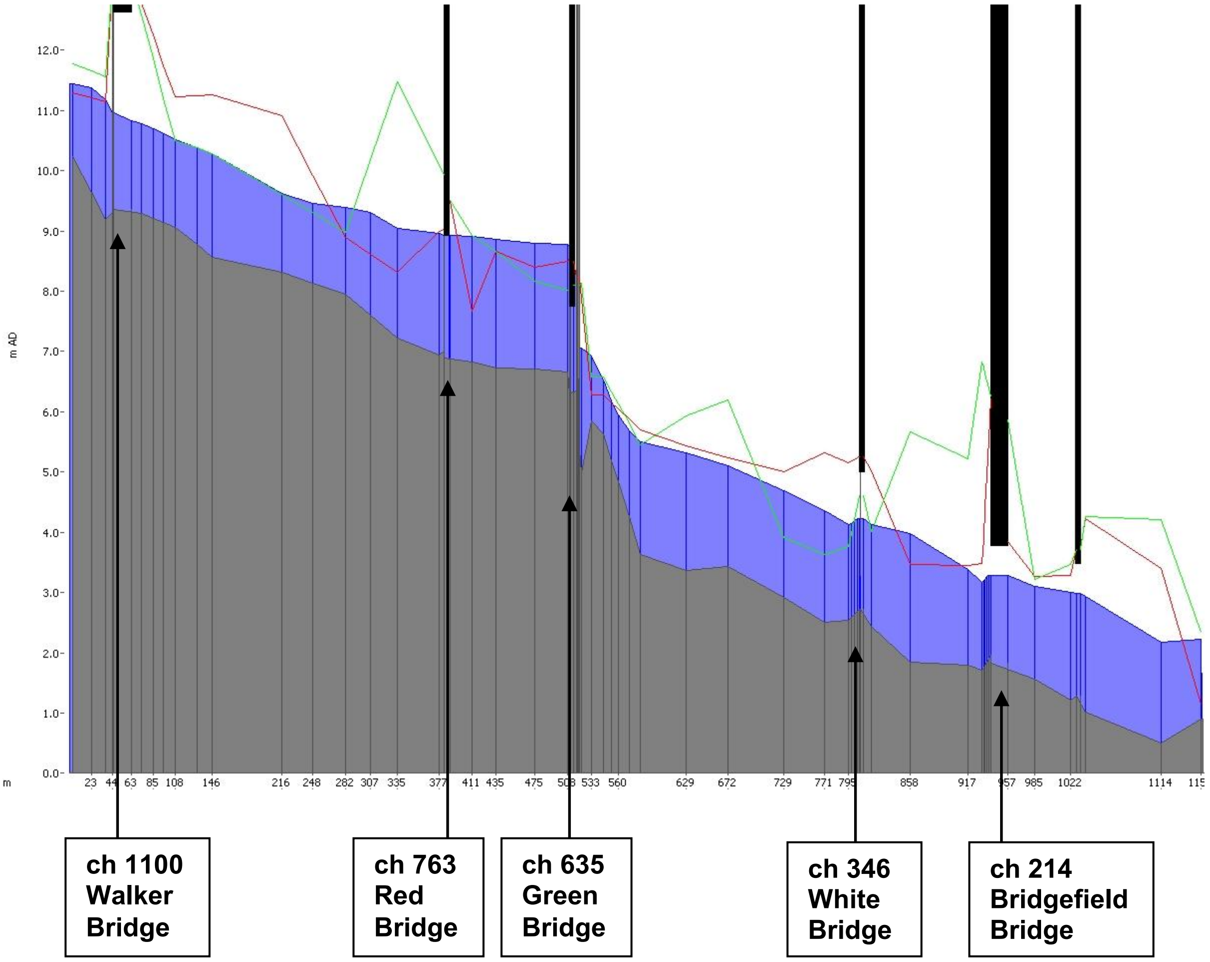
Table 3-13: Bridge Capacities extracted from the baseline model - based on water level reaching the soffit

Time	Peak Flow (m^3/s)	Peak Tide (mAOD)	Modelled Water Level at CS 40 (based on Normal depth boundary)
01 November 2009 @ 20:45	30	0.418	2.15
02 November 2009 @ 00:15	21	1.990	1.97

Figure 3-9 shows the long section output from the model for this simulation of the November 2009 event (note that the model represents the channel only). This shows that a key location of backing up and out of bank flow was above the Green Bridge and extending beyond the Red Bridge. It should be noted that the red line represents the left bank marker and the green line represents the right bank within the model, at many locations these are lower than the threshold levels set and used to assess the channel capacity.

Figure 3-9: Model Long Section showing the peak water profile for the November 2009 event (using capacity model only).

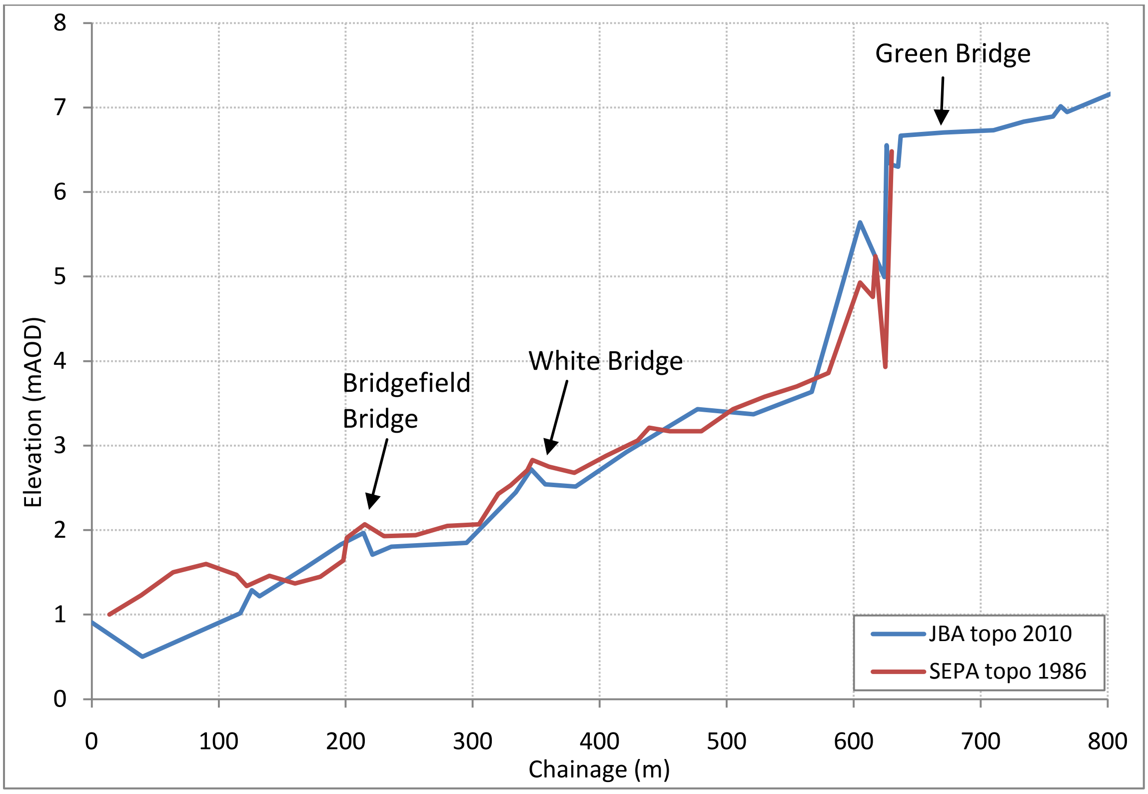
Red line - Left bank, Green Line - Right bank (model output based on Manning's n definitions rather than threshold capacities). Black vertical bars show the location of the bridge structure and lower limit of this bar indicates the soffit level.



3.5 Comparison of JBA 2010 bed level survey and SEPA 1986 survey

In 1986 SEPA carried out a topographical survey of the River Carron. Figure 3-10 below shows a comparison of this survey and JBA's recent (March 2010) survey; in general the 1986 bed levels are similar to those recorded in 2010.

Figure 3-10: Long section of channel bed levels - comparing JBA 2010 topographic survey and SEPA 1986 survey



4. Hydrology

4.1 Introduction

The catchment of the Carron Water to the Stonehaven tidal boundary covers an area of approximately 43 km^2. The Carron Water rises in low coastal hills with the highest elevation in the catchment at 321 mAOD on the Hill of Trusta. Land use within the catchment is a mixture of pasture, forestry and the urban area in the lower catchment, with the URBEXT2000 value from the Flood Estimation Handbook (FEH) CD-ROM at 0.0114[2].

The Standard Percentage Runoff for the catchment from the FEH is 37.15% and the Baseflow Index 0.581. The Standard Average Annual Rainfall is 869mm.

4.2 Data availability

The following hydrometric data was made available for use in this study:

Table 4-1: Hydrometric data

Gauge name	Gauge type	Period of record
Carron at Stonehaven	River level / flow	March 2003 - November 2009
Cheyne	Recording raingauge	April 2005 - January 2010
Mongour	Recording raingauge	October 1995 - January 2010

4.3 Rating review

4.3.1 Introduction

SEPA have an existing gauge on the Carron Water at OS NGR 8693 8565. This is not a HiFlows-UK gauge and no Annual Maximum (AMAX) or Peaks over Threshold (POT) series are available from SEPA as it is a wading gauge only. However, its 6 year record of 15 minute data and the range of gaugings available make it useful for calculating hydrological inputs to the model.

A rating review was carried out using available data. Gaugings for the Carron Water at Stonehaven were provided by SEPA for the period April 2003 to March 2010[3]. The current applicable rating equation for the gauge was also supplied by SEPA[4].

The rating equation provided by SEPA is as follows:

$$Q = 14.8469 \times (H - 0.037) ^\wedge 2.4172$$

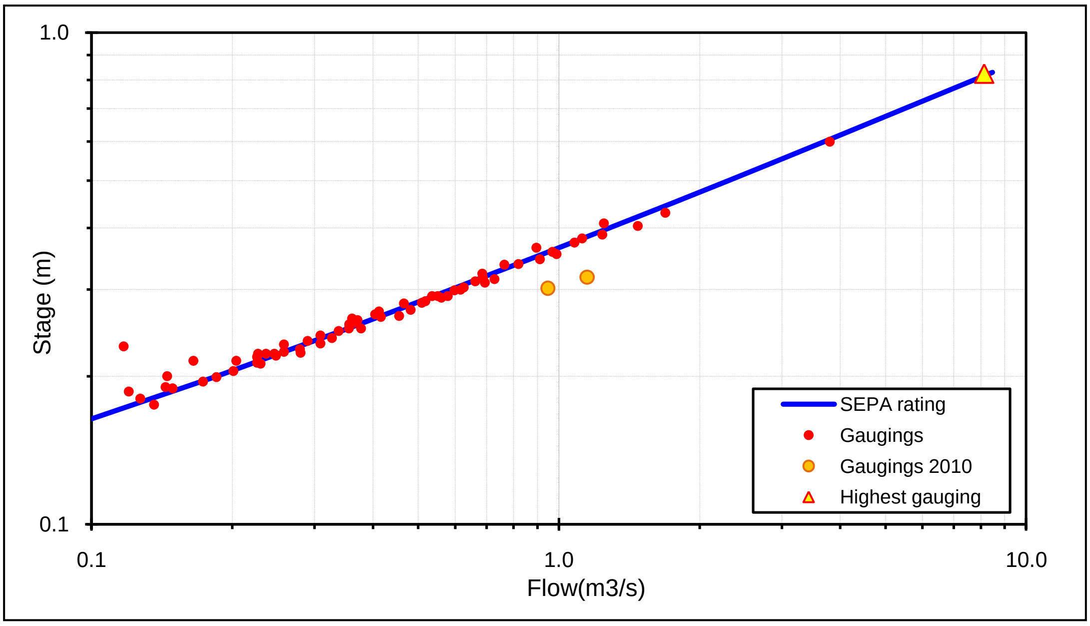
SEPA suggest that the rating is valid between stages of 0.175 - 0.6 m, although the highest gauging is approximately 0.83 m; above this any flow estimate requires extrapolation.

4.3.2 SEPA rating

Figure 4-1 below compares the rating equation with gaugings (using a log scale).

2 FEH CD-ROM version 3.
3 Email from Derek Fraser, 24 March 2010.
4 Email from Derek Fraser, 26 March 2010.

Figure 4-1: SEPA rating review



The chart shows that the rating equation compares well with gaugings taken up to September 2009. Since September 2009 two additional gaugings have been taken: in February 2010 and March 2010, which are not so well represented by the rating equation. The probable causes are the large flood event in November 2009 which resulted in a significant amount of gravel movement in the channel, and further removal of gravel in January 2010 by Aberdeenshire Council immediately upstream of the Green Bridge, both of which may have altered the bed control at the gauge.

The rating is only applicable within the range of gaugings taken to verify it, and given that this is a wading only station the highest gauging is at a level of approximately 0.83 m and 8 m^3/s. The bankfull stage at this location is c. 2.07 m, therefore 0.83 m is well within bank.

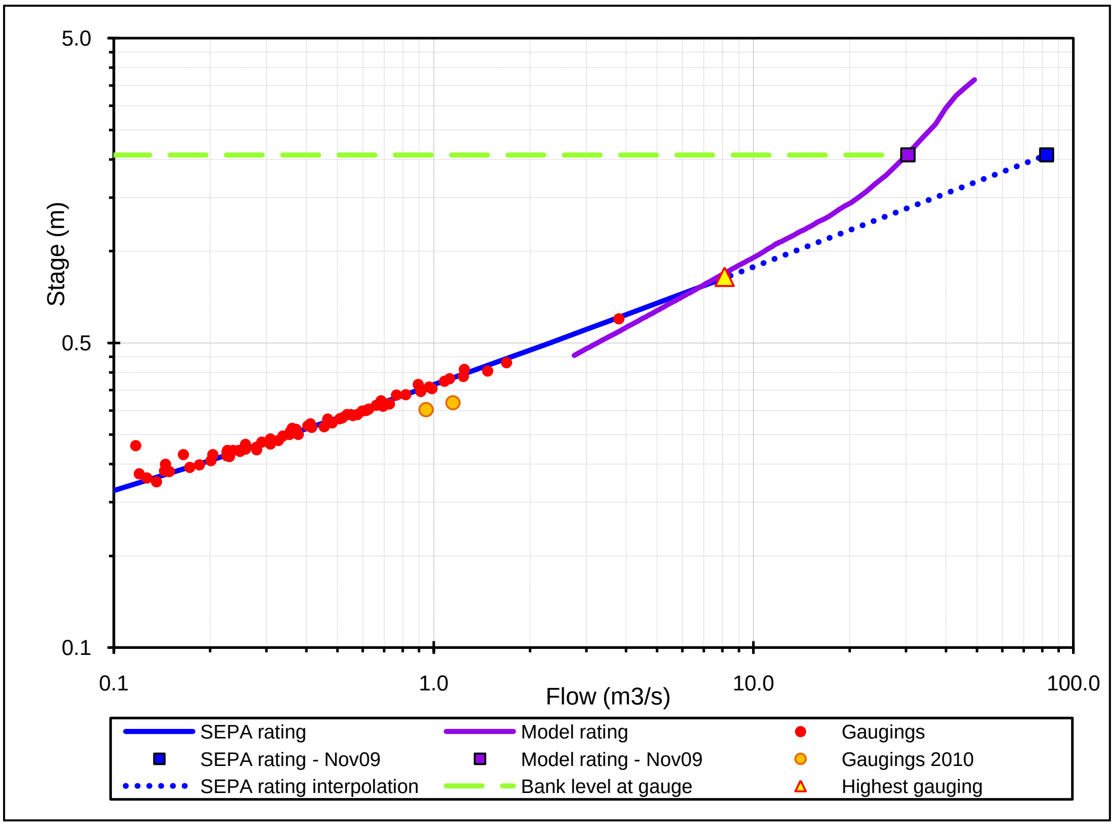
This analysis suggests that the existing rating equation may no longer be applicable to new gaugings, nor should it be used for estimating flows at high stages (this is consistent with communications with SEPA whereby the gauge was installed primarily for gauging low flows). A review of the rating may therefore be appropriate. More gaugings will be necessary in order to confirm that a change in the rating is consistent and permanent and to confirm the rating at higher river stages.

4.3.3 Model rating

The hydraulic model prepared for the Carron Water as part of this report can be used to generate a rating for comparison with that which was supplied by SEPA. As the model is based on survey taken in 2010, the model rating should reflect any change in the bed control that took place in November 2009. The model represents the channel conveyance up to bank level, beyond which extrapolation is necessary. Additional data including detailed floodplain levels would need to be included within the model to allow representation of floodplain routing and allow confirmation of the rating above bank level.

Figure 4-2 below shows the existing and modelled rating against spot gaugings, including bank level in the model and estimates of flow derived by the two ratings for the November 2009 event.

Figure 4-2: Modelled rating



This shows that the model rating is reasonably consistent with the two gaugings taken in 2010 as well as the highest gauging. The model suggests that at the gauge cross section, the November 2009 event level is approximately equal to bank level and therefore there is reasonable confidence in the rating up to this level. If the gauge was being bypassed during the November 2009 event this would affect the gauged record. The shape of the rating curve is likely to change once the river is out of bank as flow area would increase significantly for small changes in stage.

Given the stage of 2.07 m recorded in November 2009, the SEPA and model rating give very different flow estimates for this event as follows:

Table 4-2: Flow estimates for November 2009 event

Stage (m)	Rating used	Confidence in rating	Flow estimate
2.07	SEPA	Low - interpolated beyond 0.83 m	82.5
2.07	Model	Reasonable - at bank level	30.0

These flow estimates are explored further in Section 4.5.1 below.

4.4 Flow estimation method

4.4.1 Introduction

Important inputs into estimations of flood hydrology include the analysis of historical events and the estimation of flood flows for a range of annual probabilities or 'design' events. Flood estimates for catchments of this size and type are undertaken using the FEH. The FEH offers three methods for analysing design flood flows: the statistical, rainfall-runoff and hybrid methods. The statistical method combines an estimation of the median annual maximum flood (QMED) at the subject site with a growth curve, either derived from a pooling group of gauged catchments that are considered hydrologically similar to the subject site, or through single site analysis of a nearby gauge. The Rainfall-Runoff method combines design rainfall with a unit hydrograph derived for the subject site. Hybrid methods involve a combination of the two.

The FEH statistical method was deemed the most appropriate method for estimating flows on the Carron Water due to the availability of gauged data from a nearby station. The default method of flow estimation via the FEH statistical method is to derive an estimation of QMED through data transfer and then apply a growth curve for a pooling group of hydraulically similar catchments. However, in some instances where the gauged record is deemed sufficiently long a consideration of single site analysis is also viable. Both versions of the method include the use of catchment descriptors, which have been digitally abstracted from the FEH CD ROM v3 and verified through the use of OS background mapping.

4.4.2 QMED estimation

FEH guidance recommends that for gauged records of less than 14 years, QMED should be estimated using a derived Peaks Over Threshold (POT) series[5]. A threshold was applied to the gauge series to give approximately 4-5 peaks per year, along with independence criteria, in line with FEH guidance. An estimate of QMED could then be made using the equation given in the FEH Volume 3.

QMED estimates were made using flows derived from the level series using both the SEPA rating and the model rating for comparison. The estimated values for QMED are 14.7 m^3/s using the SEPA rating and 13.9 m^3/s using the model rating. Given the greater confidence in the modelled rating above the highest gauging of 8 m^3/s, the QMED estimate derived from the modelled rating will be used.

4.4.3 FEH Statistical method

Given the short period of available data the pooling group method is the most appropriate statistical approach to determining the growth curve and design flows at the site.

The FEH statistical method relies on data transfer between gauged sites and the subject site to determine the value of QMED. Given that the model uses a single inflow at the upstream boundary, and that the gauge is within the modelled reach, it is appropriate to use the QMED estimate at the gauge with aerial but no distance adjustment as a basis for calculating design flows. More detailed modelling should include separation of different inflows to the modelled reach and in this scenario a further adjustment of QMED may be appropriate.

A range of approaches were tested for creating the growth curve fittings. Given the short period for which gauged data is available, a pooling group approach is more appropriate than single site analysis but these were both undertaken for comparison. Within the pooling group approach there is the option to use an 'enhanced' pooling group, which incorporates the subject site, or an 'ungauged' pooling group, which excludes the site. Again, both these options were tested and the results compared, although the ungauged approach may be more appropriate given the short period of gauged data.

Figure 4-3 below shows the growth curves for each method tested along with the gauged data.

5 Institute of Hydrology, 1999. Flood Estimation Handbook, Volume 3.

Figure 4-3: Site growth curves

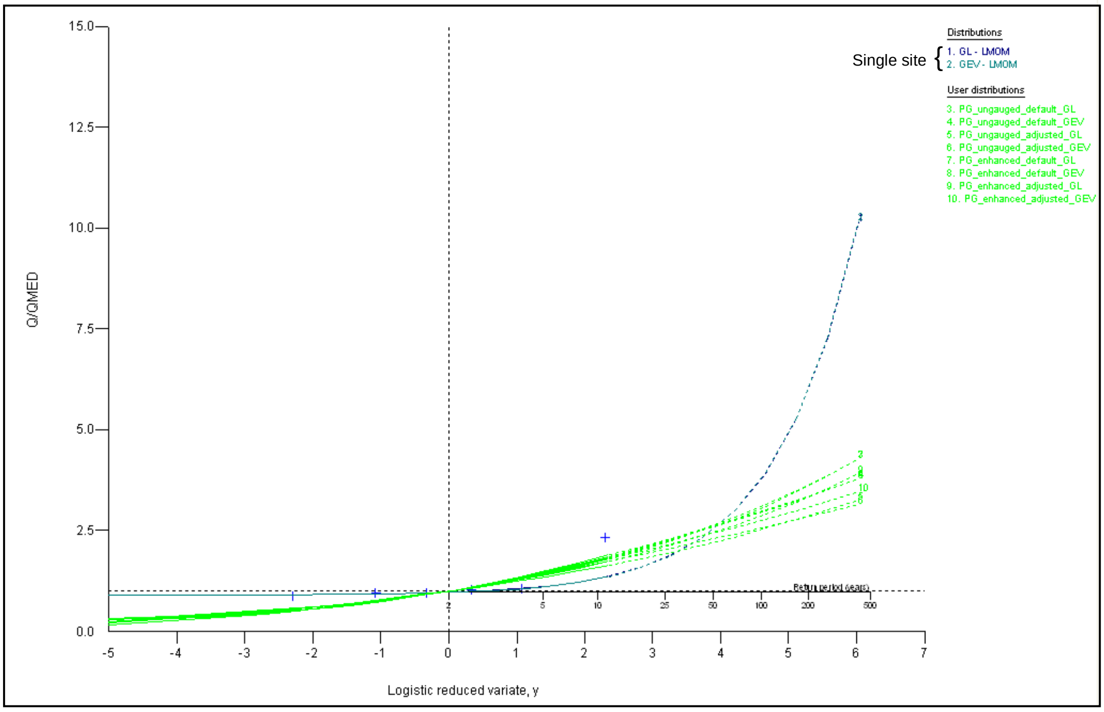


Figure 4-3 confirms that single site analysis does not give an appropriate growth curve. Figure 4-4 shows the pooled analysis results only.

Figure 4-4: Site growth curves - pooling group approach

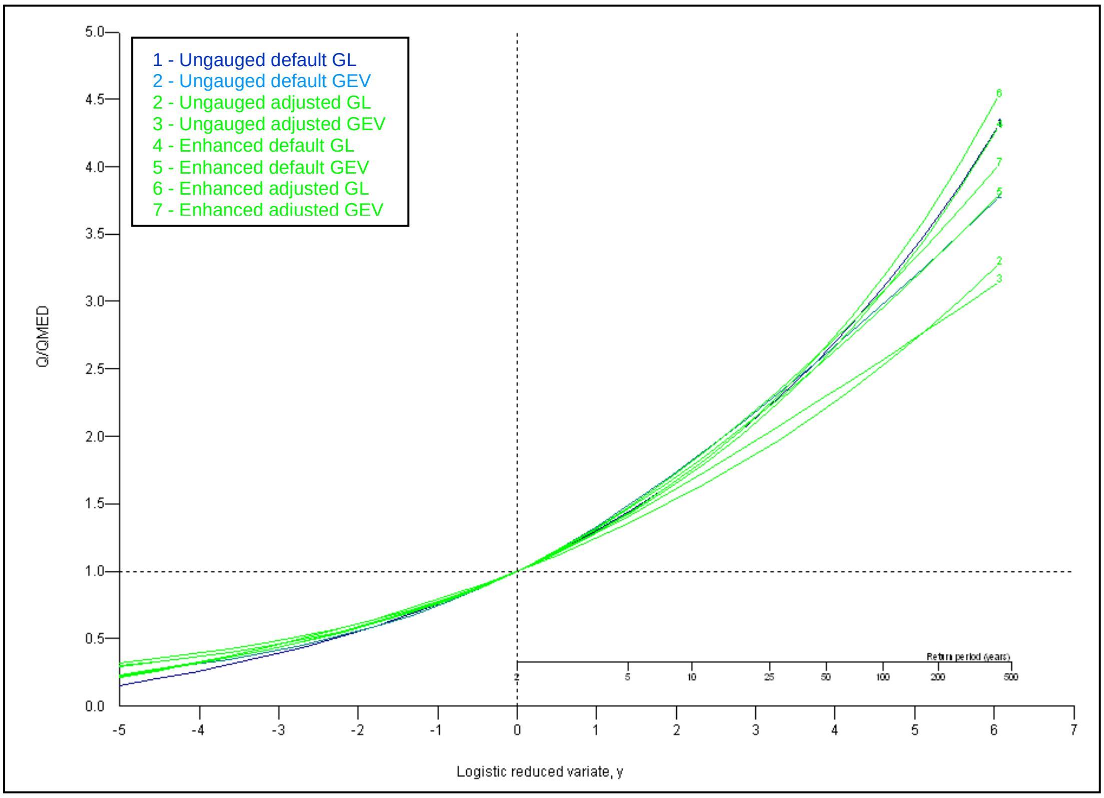


Figure 4-4 shows that the enhanced pooling approach gives slightly steeper growth curves than the ungauged approach, as to be expected given the shape of the single site distribution. However, the gauged data is not considered to offer an improvement to the derivation of the growth curve shape since WINFAP records it to be discordant and given its limited period of record. Therefore the ungauged approach will be used in preference.

The graph shows that the GL distribution gives more conservative results than the GEV distribution and given this result, and that the GL distribution is generally known to be more applicable to UK sites, this will therefore be adopted.

The adjustments made to the pooling groups included the addition of HiFlows-UK gauges falling within a 40 km radius of the SEPA gauge and removal of some discordant sites as appropriate. However this has resulted in less conservative growth curves for the ungauged, GL approach and therefore the default pooling group will be used.

Therefore the final choice of approach is the 'ungauged' site method, default pooling group and GL distribution to give a conservative yet representative growth curve.

4.5 Design flows

The design flows calculated using the method described above are show in Table 4-3 below.

Table 4-3: Design flows

Return period (years)	Annual Probability (%)	Peak flow (m^3/s)
2	50	15.4
5	20	22.1
10	10	27.0
20	5	32.2
50	2	40.2
75	1.33	44.2
100	1	47.3
200	0.5	55.4
1000	0.1	79.2

4.5.1 Historical validation

Using the AMAX series available for the gauge, and the design flows calculated above, the estimated return period of recent historical events was determined in order to help validate the design flows. The results are shown in Table 4-4. The table shows estimated flows and return periods calculated using both the SEPA and model ratings to estimate flow for comparison.

Table 4-4: Estimated return periods of historic AMAX events

Water year	AMAX stage (m)	Using SEPA rating		Using model rating	
		Flow estimate (m^3/s)	Estimated return period (years)	Flow estimate (m^3/s)	Estimated return period (years)
2003	1.05	15.3	< 2	11.7	< 2
2004	1.44	33.9	26	20.2	4
2005	1.01	13.9	< 2	11.0	< 2
2006	0.98	12.9	< 2	10.5	< 2
2007	1.01	13.9	< 2	11.0	< 2
2008	1.04	15.1	< 2	11.6	< 2
2009 (to present)	2.07	82.4	> 1000	30.0	16

Table 4-4 highlights the discrepancy between the flow estimates using the SEPA and modelled rating. The 2009 event has largely different estimated flow and hence return period depending on which rating is used; the SEPA rating suggesting a flow of approximately 80 m^3/s which equates to a return period of approximately 1,000 years, whereas the model rating suggests that the flow may be of the order of 30 m^3/s which is a return period of 16 years. This has huge implications for the probability of a flood of the same magnitude reoccurring.

Given the unreliability of the SEPA rating above the highest gauging (around 8 m^3/s) and the fact that the November 2009 modelled stage is around bank level, below which there is reasonable confidence in the modelled rating, it is fair to assume that the estimate of flow and return period based on the model results is more reliable (up to a flow of 30 m^3/s).

This could be confirmed by obtaining gaugings at higher river levels although it is understood that the gauge is currently a wading only station[6].

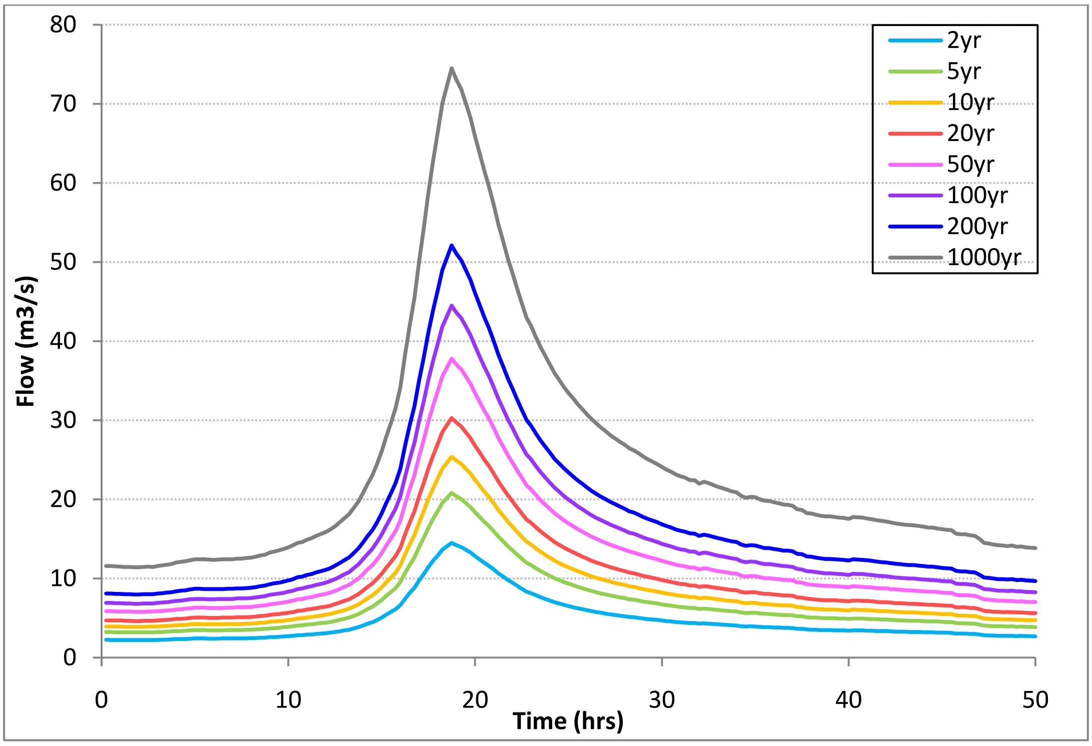
Between 2003 and 2008, 5 out of 6 events have an estimated return period of less than 2 years and one has a return period of 4 years. These are reasonable results for this period of data.

4.6 Hydrograph shape

For the purposes of hydrodynamic modelling, a representative hydrograph shape as well as design peak flow is required. For this study a standardised hydrograph was synthesised from the gauged record using an average of all single-peaked hydrographs of a significant size. This standardised shape was then scaled to the design flows given in Table 4-3 above to give design hydrographs, as shown in Figure 4-5 below.

[6] Email from Derek Fraser, April 2010.

Figure 4-5: Design hydrographs based on synthesised shape



5. Hydrometric data analysis - discussion of the November 2009 event

5.1 Nature of November 2009 event

The November 2009 event is known to be unprecedented in terms of its magnitude but it is also useful to consider other aspects of the event hydrograph shape.

Figure 5-1 below compares the November 2009 hydrograph shape with those of the other 6 AMAX events recorded by the gauge. Table 5-1 below compares the time to peak and total volumes of these events.

Figure 5-1: AMAX hydrographs comparison

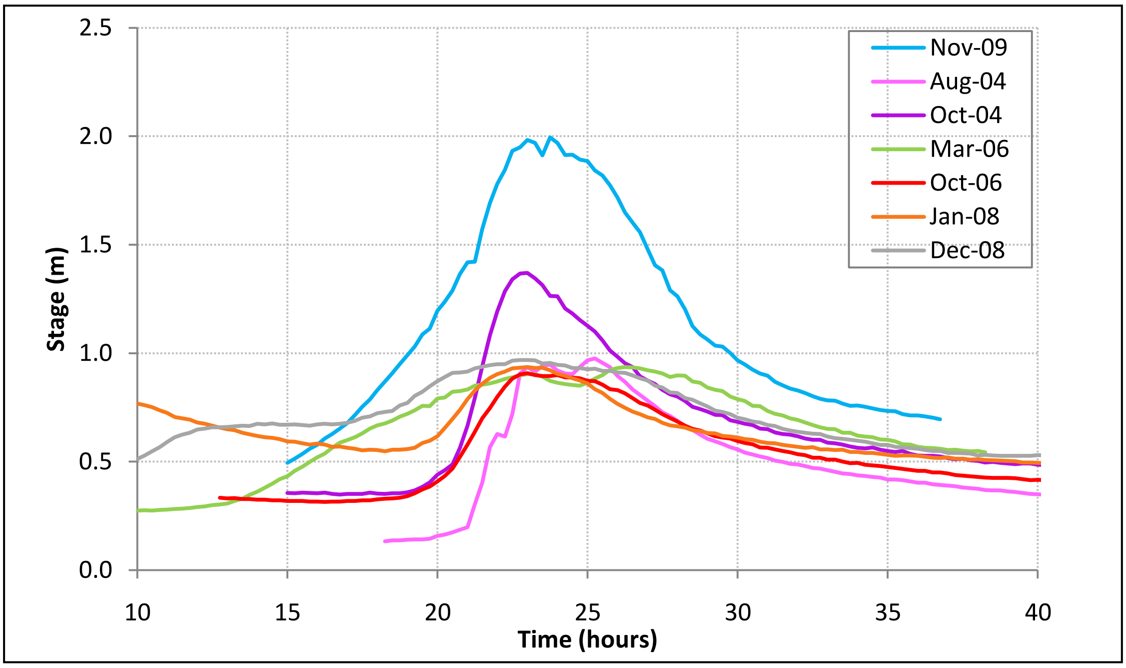


Table 5-1: AMAX hydrographs comparison

Water year	Peak flow (m3/s)	Time to peak (hours)	Hydrograph volume	
			Total volume (m3)	Time period for total volume (hours)
2003	15.3	1.75	410,000	24.25
2004	33.9	3.75	770,000	22.75
2005	13.9	12.25	720,000	27.50
2006	12.9	5.75	470,000	28.00
2007	13.9	4.75	610,000	31.75
2008	15.1	14.50	1,100,000	47.25
2009 (to present)	82.4	6.00	2,150,000	19.75
Note: Flows and volumes based on SEPA rating, providing relative comparison although absolute values may not be representative.				

This data suggests that as well as being a much larger peak flow than any of the other events, the 2009 event had a relatively short time to peak. Therefore the large total volume was concentrated within a relatively short space of time and the river would have experienced a rapid rate of rise.

5.1.1 Comparison of the November 2009 event with events in January and February 2010

Table 5-2 below shows a comparison of the gauged stage and flows from the 3 significant events in 2010 and event in October 2009 compared to the November 2009 event. This suggests that the 2009 event was substantially larger, matching observations that it caused significantly more flooding.

Table 5-2: Magnitude of 2009 and 2010 events

Date	Time	Stage @ Gauge	Flow (SEPA rating)	Flow (model rating)
22/10/09	02:30	1.257	24.0	16.2
01/11/09	20:45	2.068	80.0	30.0
16/01/10	13:00	1.062	15.8	11.9
25/02/10	15:00	1.038	14.9	11.5
26/02/10	06:15	1.055	15.5	11.8

Note that the model rating is based on March 2010 survey data and therefore reflects the channel geometry at this time. The SEPA rating matches the pre-2010 gaugings at low flows and therefore the previous channel control. However, at the relatively high stages of these events the difference in the results is not a factor of sediment removal but rather that the SEPA rating is interpolated well beyond the maximum gauging.

5.2 Comparing the Carron Water and River Bervie

5.2.1 Correlation between peak stages

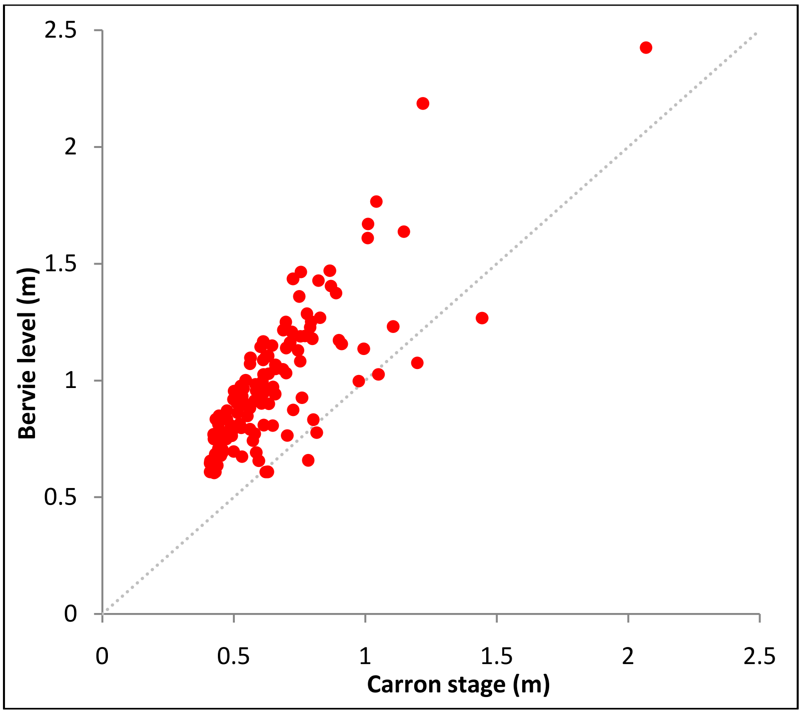
The River Bervie catchment is a major catchment south of the Carron Water. The Bervie and Carron Water are believed to display similar responses to flood events and given their close proximity, are likely to experience similar rainfall. Therefore it is useful to compare gauge data from each catchment to determine if there is a correlation and if so, what the magnitude of the November 2009 event on the Bervie was, to provide another point of reference.

The Bervie is gauged at Inverbervie (HiFlows-UK Reference no. 13001). Gauge data was provided by SEPA for the period 1979-2010[7]. The overlapping period of data between the Bervie and Carron Water is therefore 2003-2010. 125 peaks were extracted from the Carron record (based on a stage threshold of 0.4 m) and plotted against the peaks for the same events gauged at Inverbervie (note that the Bervie catchment here is more than double the size of the Carron Water at Stonehaven and therefore peaks on the Bervie are generally slightly larger and occur slightly later).

Figure 5-2 below shows the peak stages for these 125 events on the Carron Water and River Bervie plotted against each other (note that stage has been used given the uncertainty over the Carron Water rating). It is clear that there is a relatively strong correlation between the two and therefore the Bervie provides a useful comparison.

[7] Information received from Derek Fraser, SEPA. May 2010.

Figure 5-2: Correlation between Carron Water and River Bervie peak stage



5.2.2 Similarities in hydrograph shape

As noted above, the peak stage for individual events on the Carron and Bervie show a strong relationship. However, it is also useful to examine the similarities in hydrograph shape and hence the river response to determine how closely the two series can be correlated.

The largest events within the overlapping period of data were examined to determine similarities between the Carron and Bervie stage series; see Figure 5-3 to Figure 5-5 below. This suggests that there is also a strong correlation in the hydrograph responses at the two gauges.

Figure 5-3: October-November 2009

(1st, 3rd and 5th largest events within overlapping period)

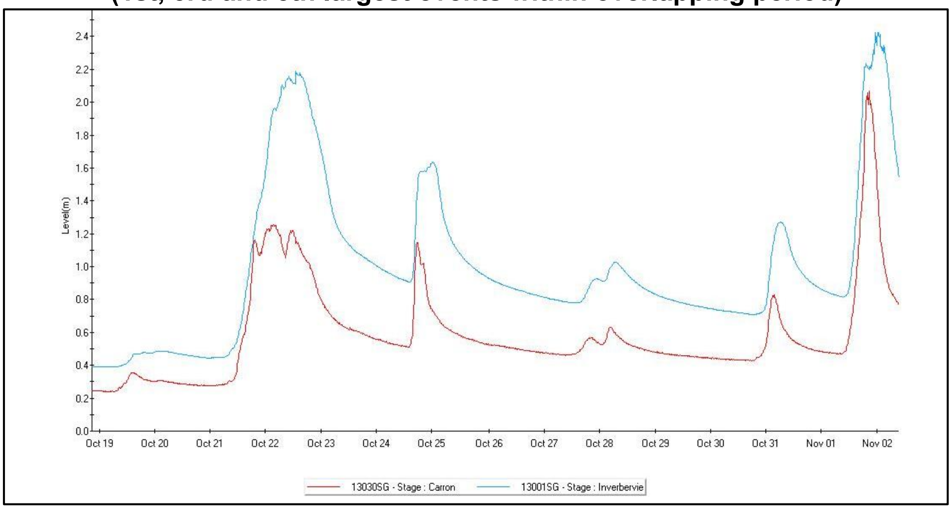


Figure 5-4: October 2004
(2nd and 6th largest events within overlapping period)

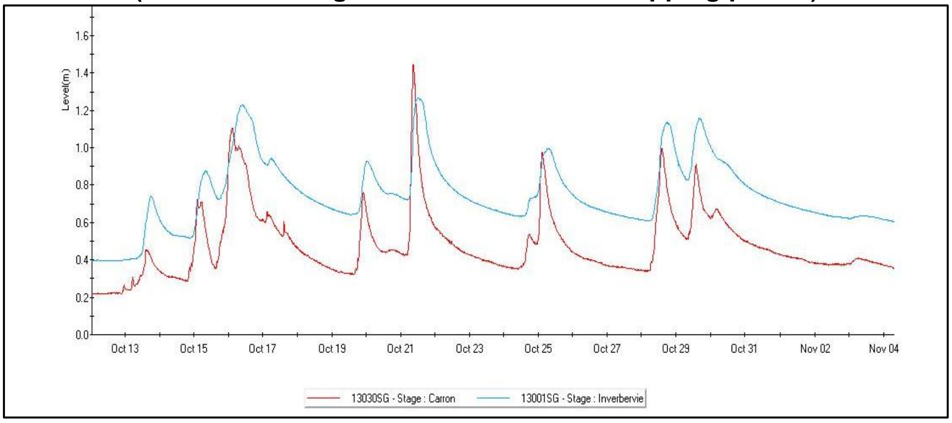
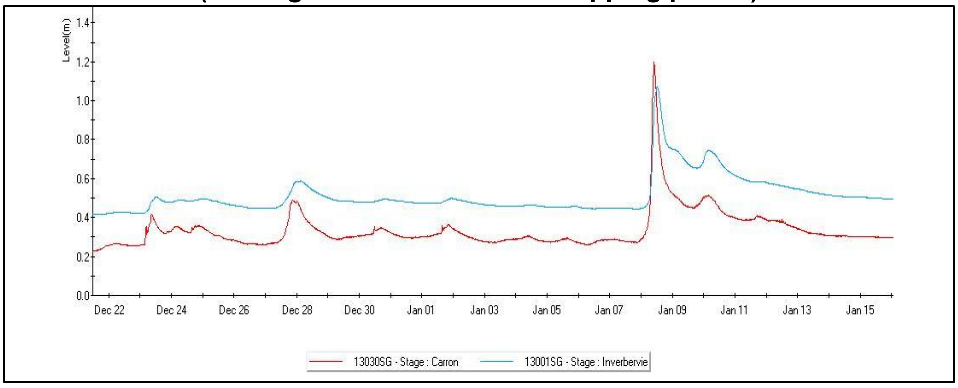


Figure 5-5: January 2005
(4th largest event within overlapping period)



5.2.3 Magnitude of the November 2009 event on the River Bervie

Using the FEH Statistical method, an estimate of the return period of the November 2009 event on the River Bervie can be derived. Pooling group analysis was used to estimate growth factors based on gauged data from the Bervie at Inverbervie, and thus to derive a growth curve using the QMED estimate from the AMAX series.

The November 2009 event at Inverbervie has an estimated flow of 67.1 m^3/s based on the existing rating, which according to the single GL site growth curve based on 31 years of record equates to a return period of approximately 23 years. [Using the pooled growth curve this equates to return period of approximately 40 years]. This is a slightly higher estimated return period than that of the same event on the River Carron using the modelled rating, but is of the same order of magnitude, suggesting that this estimate is more reliable than that derived from the SEPA rating.

5.3 Rainfall data for the Carron Water catchment

5.3.1 Local rain gauges

Rainfall records for the Cheyne and Mongour recording rain gauges were provided for use in this study. The location of these gauges is shown in Figure 5-6 below.

Figure 5-6: Rain gauge locations

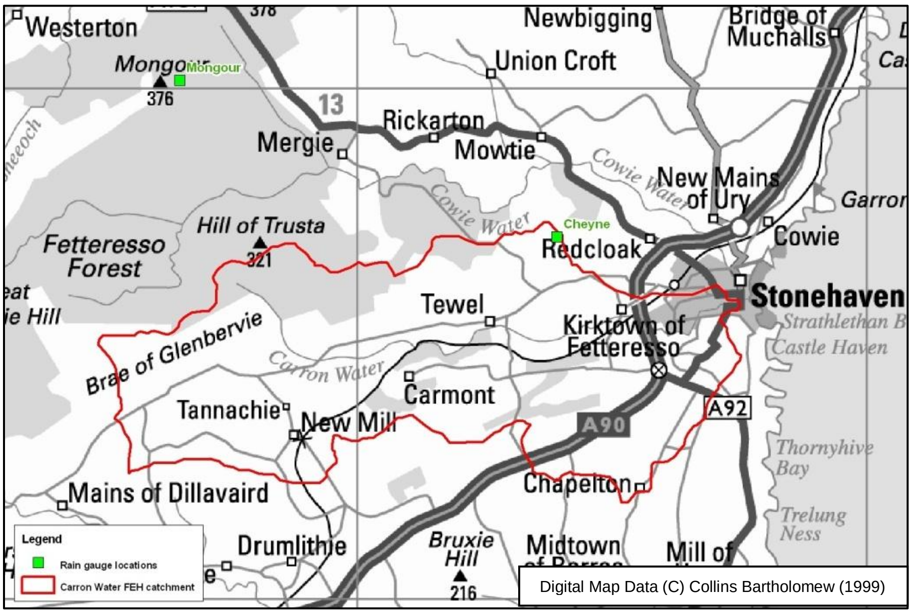


Figure 5-6 suggests that the Cheyne and Mongour rain gauge locations are reasonably representative of the catchment, with Mongour located on higher ground similar to the upper Carron Water catchment, and Cheyne on the edge of the middle/lower catchment.

This suggests that should a forecasting model be developed for the Carron Water in the future, these two existing gauges would be expected to form a reliable input.

5.3.2 Comparison of event rainfall and river stage

Rainfall records can be compared to the gauged record for the Carron Water to provide an indication of how closely they are related and whether the rain gauges would be suitable for the provision of flood warning.

November 2009

A review of the hydrometric data in November 2009 shows that both Cheyne and Mongour rain gauges recorded a relatively high and sustained amount of rainfall in the period leading up to the peak flow recorded in the Carron Water. Peak rainfall rates are gauged at 3-4 mm/15min at Mongour. See Figure 5-7 below. There was also a substantial amount of rainfall in the period between mid-October and the start of November with two other minor events during this period and causing wet antecedent conditions which would have exacerbated runoff and peak flows in the November event. See Figure 5-8 below.

Figure 5-7: Rain and river gauge records for the November 2009 event

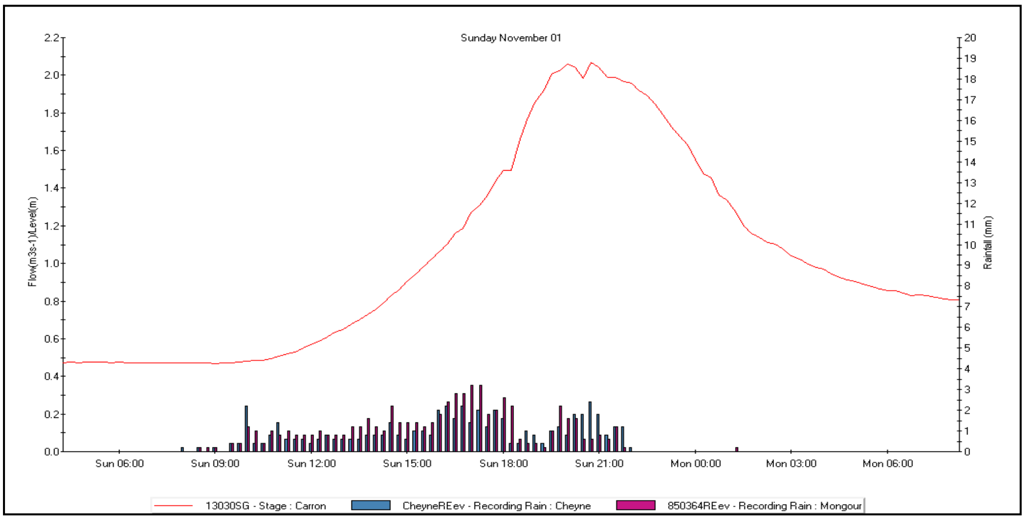
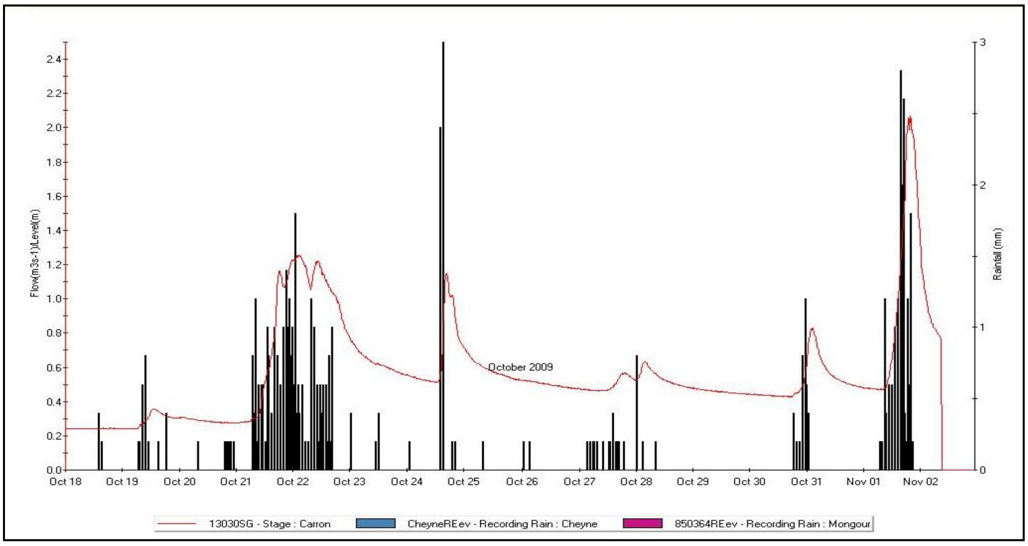


Figure 5-8: Rain and river gauge records for October to November 2009



The Depth-Duration-Frequency (DDF) modelling function within the FEH CD-ROM can be used to give an estimate of the return period of the rainfall recorded at the two gauges. This analysis looks at the individual rainfall event but does not consider the antecedent conditions. The following rainfall return periods were derived for the October 2009 and November 2009 events in isolation, as well as a return period for their combined rainfall.

Table 5-3: Estimated rainfall return periods for October-November 2009

Event dates	Rainfall gauge	Total rainfall (mm)	Time period for rainfall (hours)	Estimated rainfall return period (years)
19/10/2009 - 01/11/2009	Cheyne	203.2	328.5	119.0
	Mongour	274.6	328.25	239.0
21/10/2009-22/10/2009	Cheyne	86.0	36.0	13.7
	Mongour	124.0	36.75	79.0
01/11/2009	Cheyne	54.0	14.25	7.8
	Mongour	64.6	13.5	23.0

This suggests that although the November 2009 event has a relatively low rainfall return period, the combined return period for the period between the October and November 2009 events is extreme, at over 0.5% AP (200 year) for Mongour and over 1% AP (100 year) for Cheyne. Therefore to have heavy rainfall as experienced in November 2009 on top of such wet antecedent conditions is a relatively low probability event.

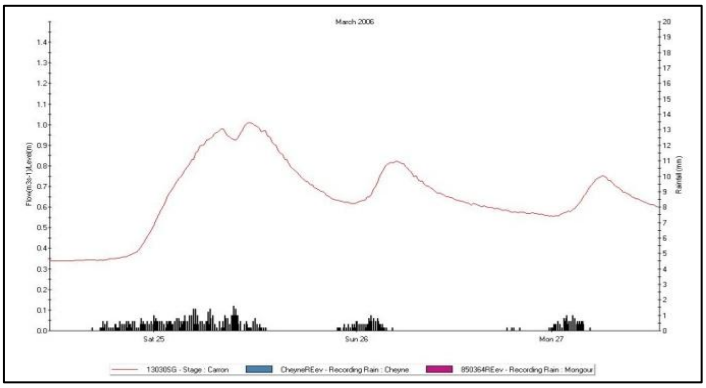
Other AMAX events

Rain gauge and river gauge data are also available for other AMAX events and again show strong comparability in the amount and pattern of rainfall and river levels. See Figure 5-9 below.

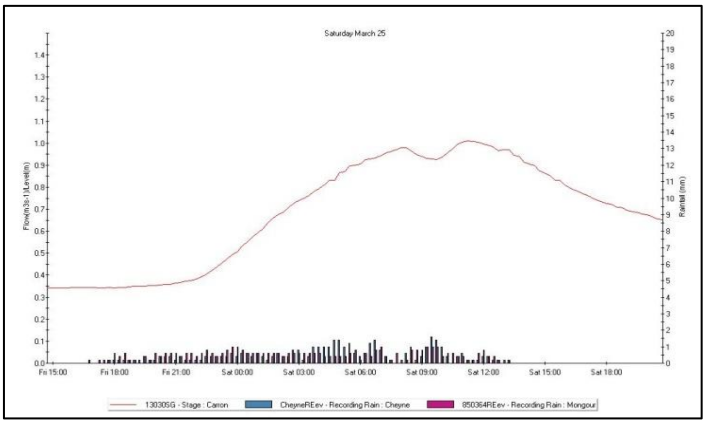
It can be seen that rainfall depths are significantly lower, of the order of 1-2 mm/15min, for AMAX events other than November 2009.

Figure 5-9: Rain and river gauge records for other AMAX events

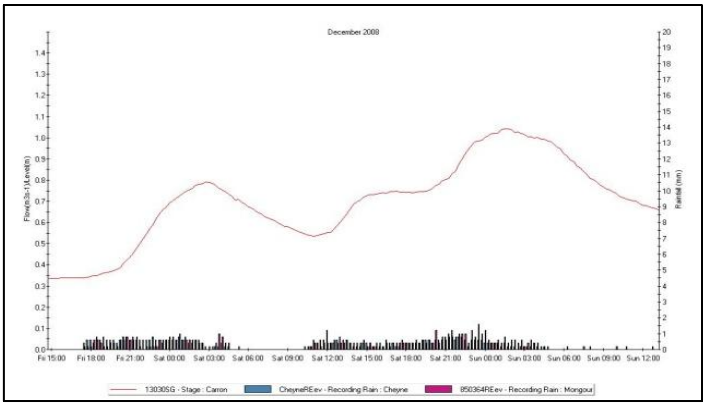
March 2006



January 2008



December 2008



6. Conclusions and Recommendations

Following significant flooding to Stonehaven in November 2009, JBA were commissioned in March 2010 to carry out topographical survey of the watercourse and report on the capacity of the channel. This report therefore serves as a capacity check of the River Carron from Walker Bridge to the North Sea. The capacity check is based on topographic survey collected in March 2010.

The design flows detailed within this assessment have been derived using standard Flood Estimation Handbook (FEH) methodologies for the model downstream boundary; therefore including any inflows from tributaries of the River Carron, including the Glaslaw Burn. As part of this study an analysis of available hydrometric data has been undertaken. Due to the nature of the existing gauging station being suitable for the assessment of low flows only the large uncertainties when deriving design flows for this particular reach have been demonstrated. Analysis of the River Bervie data indicates a potential correlation between the peaks on the River Carron and Bervie. This data could be further analysed at the next stage with the aim of improving the uncertainties in design flows and return periods. Peak flows on the Glaslaw Burn would also be calculated at the next stage.

Furthermore the bed of the channel would appear to be constantly changing in terms of sediment transport and disposition, in particular during and immediately after flood events. Any future design of mitigation/ capacity improvement works would need to consider the dynamic nature of this system, climate change and geomorphological processes.

The lowest channel capacity resulting under the current conditions is that of the channel immediately upstream of the Green Bridge. At this location water levels are controlled primarily by the 'log weir' at low flows and then as water levels increase the framework on the underside of the Green Bridge also contributes to a reduction in capacity. At this location the channel capacity is currently in the region of 20 m^3/s (estimated in this study to have a return period of approximately 5 years).

The capacities of each of the main bridges have been extracted from the baseline model and this initial modelling exercise would suggest that the Walker Bridge, White Bridge and Bridgefield Road bridge have capacities of 50 m^3/s or greater and that the Green Bridge has the lowest capacity at 15.8 m^3/s. However discussions with Steve McFarland indicated that anecdotal evidence provided to the council suggested that the Bridgefield Road bridge was running full or even backing up during the November 2009 event, further analysis of the Bridgefield Road bridge may therefore be required.

A number of scenarios capacity check scenarios were undertaken as part of this study:

- Scenario 1 - Assessment of the effect of the island structure & rock armour on the left bank next to sewer and removal of log weir
- Scenario 2 - Assessment of the capacity restriction posed by the underside of the Green Bridge
- Scenario 3 - Assessment of the capacity of Green Bridge prior to sediment removal
- Scenario 4 - Assessment of narrowing of the channel at Green Bridge
- Scenario 5 - Test the capacity of the lower reach using low tide level & 200 year extreme sea level
- Scenario 6 - Assessment of removing the log weir and raising trellis framework underneath the Green bridge combined
- Scenario 7 - Assessment of the garden section from the White Bridge to the Sea

The greatest increase in capacity was found by reducing the level of the 'log weir'; by reducing the level of the weir to match that of the upstream bed level (a reduction of approximately 0.4 m) this resulted in an increase in capacity at Cross Section 637 (the location of lowest capacity in the base model run) by 7.2 m^3/s, increasing the overall capacity from 20 to 27 m^3/s (estimated in this study to increase return period from approximately 5 years to 10 years).

Should the model be used for further detailed analysis and optioneering, it is recommended that the model be extended to include out of bank flow areas (in particular within the reach from Red Bridge to Green Bridge). A detailed model of the reach would also look to include inflows from individual watercourse such as the Glaslaw Burn. Further analysis could also be carried out to take into account the effect of the A90 culvert upstream of the study reach on peak flows using a routing model.

Any future modifications made to improve the capacity along the reach, including that at the Green Bridge, would need to demonstrate that they do not increase the pass forward flow and hence increase flood risk further downstream.

Due to the topography of Stonehaven it would also be advised that to enable a full assessment of flood risk to the town to be undertaken, it would be prudent to evaluate the risk from surface water flooding and the drainage network.

Improvement of direct defences (existing walls) and demountables through the town may be possible and one potential option may be to store water further upstream in the catchment and to then only allow the flow equivalent to the lowest capacity within the main settlement to pass forward below the storage area, this would be in the region of 20 m^3/s. This could be further investigated at the next stage.

Figures

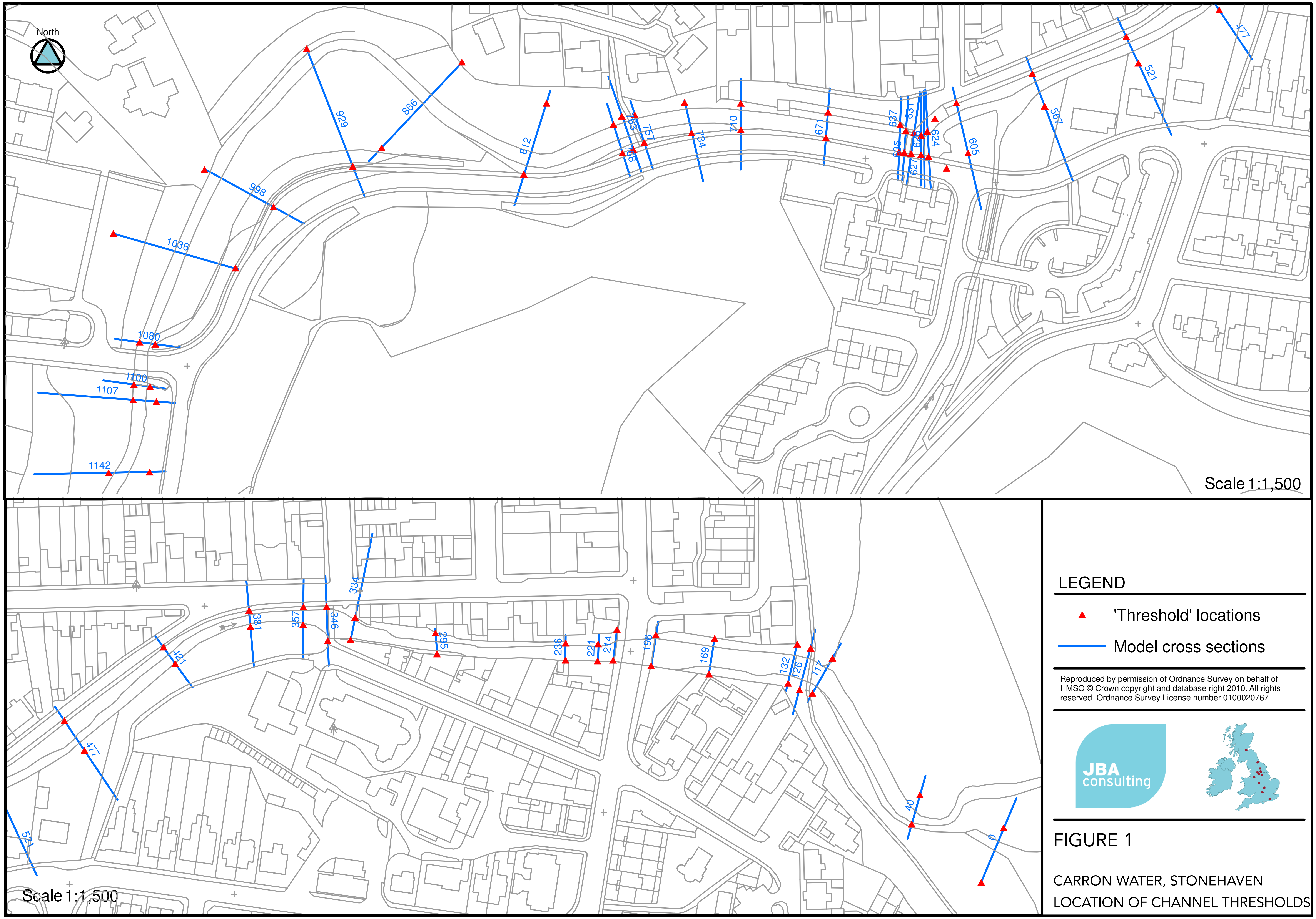


FIGURE 1

CARRON WATER, STONEHAVEN
LOCATION OF CHANNEL THRESHOLDS

Offices at

Atherstone
Doncaster
Edinburgh
Haywards Heath
Limerick
Newcastle upon Tyne
Newport
Northallerton
Northampton
Saltaire
Skipton
Tadcaster
Wallingford
Warrington

Registered Office
South Barn
Broughton Hall
SKIPTON
North Yorkshire
BD23 3AE

t:+44(0)1756 799919
e:info@jbaconsulting.co.uk

Jeremy Benn Associates Ltd
Registered in England 3246693